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The Annual Subscription, post free, to any part of the world is 21/- (English) or its equivalent in other currency. The price of single copies is 1/6, or 1/9 if sent by post. Back numbers (when available) are charged double if over six months old.

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The Dock & Harbour Authority

No. 337. Vol. XXIX. NOVEMBER, 1948

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Editorial Comments

The Panama Canal.

In this, and a subsequent issue, we are publishing an abridged paper by James H. Stratton, M.A.S.C.E., and précis of other papers from the Proceedings of the American Society of Civil Engineers describing the investigations, studies and estimates of costs of the means for increasing the capacity and security of the Panama Canal to meet future needs, which were ordered by the American Senate and House of Representatives and approved by President Truman on December 28th, 1945.

The Future and the Panama Canal is printed almost intact. It is regretted, however, that owing to space restriction it is possible only to give brief synopsis or précis of the mass of data and information which forms the remaining papers.

These précis present in handy form a comprehensive review of the broad basis of the investigations, the methods of study adopted, and the many and various technical and engineering features of the studies. The Foreword prepared contains a short description of the Panama Canal from the Middle Ages which, it is thought, will form an interesting approach to a study of the subject.

It will be seen that the investigations described in the Paper and précis centre around the scheme for conversion of the present Panama Canal to a sea-level canal.

However, since the investigations and the report to Congress based thereon was made, recent developments in the political situation in Central America has somewhat altered the outlook of the United States Government.

Rejection by the National Assembly of the Republic of Panama of a proposed agreement between that Republic and the U.S.A. as to the formation of military bases for defence purposes in the present Panama Canal zone has once again brought into prominence the vital question as to whether a second route for another canal should not be sought elsewhere than in the Panama Canal zone.

As far as it has been possible to obtain reliable information, the present position appears to be that certain of the alternative canal routes mentioned in Mr. Stratton's paper have been revived for further study. These routes apparently are the one across Mexico from Tehuantepec to Puerto Mexico, that across Nicaragua from Brito through Lake Nicaragua to Greytown, and another across Columbia joining the Atrato and San Juan Rivers.

The Mexican route, if it follows the alignment mentioned in Stratton's paper, is shown to be a lock canal 165 miles in length.

The Nicaraguan route is also a lock canal project, 173 miles long; there would be, however, only two locks—between the Lake and the Pacific. The idea of this canal route is not new, for although the United States proceeded with the Panama Canal construction, the Bryan-Chamarro Treaty was signed in 1916 by the terms of which the U.S.A. paid to Nicaragua three million dollars for an option to construct a canal, which presumably still stands.

The Colombian proposal appears to be the route suggested by Enrique Ancizar in 1930 and involves some deep excavation across the foothills of the Andes to unite the head-waters of the two rivers. Besides being a sea-level route, it would be a long fresh water course. The Governments of both countries are studying this proposal and its implications.

Whichever route is finally selected, the necessity for a second sea route from the Atlantic to the Pacific is fully realised in the U.S.A., and no doubt a decision will be arrived at without undue delay.

We feel, therefore, that what we have been able to present of the Papers on the Panama Canal will be of considerable interest to readers, both in respect to the lines of approach by the U.S.A. to the study of canal routes, the technical data obtained, the conclusions arrived at, and as indicating the general trend of thought in the United States in regard to the pressing need for improving the sea route across Central America either by alternative routes for a new canal or conversion of the present canal into a sea-level canal.

Land Reclamation.

The increase in population and expectancy of life of most of the inhabitants of the world, together with shortages of food, due to the war years, coupled with loss of fertility in the soil and soil erosion, has focussed attention upon the urgent necessity for increasing the productivity of the earth's surface.

This desirable result can be attained, it is hoped, by scientific farming and fertilization methods, improvement of seed, and the return to fertility of areas of exhausted soil and cultivation of new territories in various parts of the world. Lastly, but naturally of somewhat less importance, reclamation of land—marshes and areas of saltings and mud flats from the sea and the estuaries of rivers—can be regarded as a contribution to the problem.

It is therefore with interest that we learn of a scheme of land reclamation which has been undertaken by farmers in the parishes of Friskney and Wainfleet St. Mary, bordering on the Wash and lying south-west of Gibraltar Point, and which is now reaching a crucial stage in its construction.

The scheme in itself is not on a large scale, but since the beginning of the year contractors, on behalf of seventeen "Riparian" farmers, have been building a new sea-wall nearly six miles long to exclude the tides from roughly 1,400 acres of saltings or salt marsh.

The sea-wall has been constructed along the whole frontage, about half a mile seaward of the present sea defence wall, built in 1810 under powers given by an Act of Parliament, and in most

Editorial Comments—continued

places has reached its full height. In some places, however, finishing protective work still has to be completed before the race against the high equinoctial tides of the year can be claimed to have been won.

Dykes are being cut for drainage and much levelling and filling up of creeks and freeing the soil from excessive salt will have to be carried out before cultivation is practicable. It is anticipated, however, that ploughing of the reclaimed land will be possible next summer and that tne area will be cropped early in 1950.

Similar land reclaimed many years ago now yields 10 tons of potatoes and six to seven quarters of wheat to the acre, so that the economics of the venture are beyond question.

It is hoped in a later issue to be able to report in detail the civil engineering and other aspects of the reclamation work involved.

It is our view that there are possibilities in the systematic reclamation of large areas of land in the many river estuaries of the British Isles. In Holland the Dutch, who, of course, are the world's experts in such work, have in hand a great reclamation scheme, much of it now completed in the Zuyder Zee.

One would have to proceed warily in the case of river estuaries such as the Wash, the Thames or the Mersey, if large reclamations were contemplated, in order that the areas involved, however valuable from a food productivity point of view, would not upset the regimen of the river and estuaries concerned. On the other hand, it is probable that reclamation work, if scientifically investigated and carried out, might improve the self-maintenance of navigable channels. The use of hydraulic models for these experiments might also be helpful.

It is doubtful, however, whether the Parliamentary and other powers at present possessed by Port and Harbour Authorities would enable such work for purely reclamation purposes to be undertaken and financed by them. It therefore appears that reclamation of land on the scale as envisaged might well have to be, at the outset, the subject of a preliminary enquiry by a Government Select Committee. Later it would seem to follow normally that the scientific proposals would be investigated by the new Hydraulic Research Board, which has recently been set up by the Department of Scientific and Industrial Research.

An increasing interest is being taken by civil engineers in the use of models for estuarial experiments, and in the near future we hope to publish further articles on the subject.

Valuable Engineering Contracts for Britain.

It is announced from Buenos Aires that the Argentine Government has signed contracts valued at about £2,000,000 with two prominent British engineering firms for the complete mechanical and electrical equipment of a 150,000-ton grain storage and shipping elevator at the Port of Buenos Aires. The first contract is with Messrs. Henry Simon, Ltd., of Stockport, one of the leading international firms of flour milling and grain handling engineers, and the second order has been placed with the General Electric Co., Ltd., of London, who will supply the electric motors, transformers, main switch gear, lighting equipment and telephones.

This is the fifth large terminal elevator to be built and equipped under the Argentine Government's far-reaching national grain elevator scheme. The first four elevators, also equipped by the same firms, are now in operation at the Ports of Bahia Blanca, Rosario Sud, Quequen and Villa Constitucion, and the concrete buildings for the new Buenos Aires plant are already in existence, having been built at the same time as those of the four elevators mentioned above. The work of designing the equipment is already far advanced so that much of it can be put into fabrication at once. The bulk of the machinery will be manufactured in Britain, but arrangements have also been made for some of the equipment to be supplied locally.

The new elevator, which will be the largest in the southern hemisphere, will be able to receive grain from railway wagons at the rate of 2,000 tons per hour and load it in bulk into ocean-going ships at the rate of 3,000 tons per hour; there will be berthing and loading facilities for five ships at a time. Means will also be provided for receiving grain from barges and road vehicles, and the equipment of the elevator will include grain drying and cleaning machinery.

A third contract worth over £400,000 has been placed by the Government of Pakistan with Messrs. Ransomes & Rapier, Ltd., of Ipswich, for water control gates, operating gear and other equipment for the new Barrage at Kotri on the River Indus. Kotri is about 200 miles downstream from Sukkur, the site of the famous Lloyd Barrage for which the same firm supplied the gates and equipment nearly 20 years ago.

It is encouraging to learn that these three British firms have been able to secure these valuable contracts in spite of strong foreign competition. It also is an excellent advertisement for British industry and enterprise.

Ship Fires and Explosion Hazards in Ports.

In the Editorial columns of our last issue we referred to the Working Party set up by the Ministry of Transport to consider the problem of Fire Prevention and Fire Fighting in Ships in Port. The Ministry now announce that the composition of the Working Party has been completed with the inclusion of the Officers' Merchant Navy Federation and that the first meeting has been held.

In connection with the bulk storage and handling of Ammonium Nitrate and the explosions at Texas City and Brest also mentioned in our last issue, we learn that the experiments referred to were duly carried out between September 29th and October 2nd, on the Island of Dune, Heligoland, on behalf of a working party appointed by the Home Secretary.

The object of the experiments was to ascertain whether pure ammonium nitrate when stored in bulk would detonate when subjected to intense and prolonged heat. Three large-scale trials were carried out, one in a bunker and two in barges, involving in all some 240 tons of ammonium nitrate. In the bunker trial the ammonium nitrate was stored in drums; and in the barges one part was stored in drums and one in paper bags. In no case was there any evidence of detonation or explosion taking place.

Up to the present we have not been informed as to the nature of the tests carried out nor as to the details of any precautionary measures which it may be considered desirable should be adopted in the handling and storage of the material.

In any case, however, the results appear to have been contrary to what was expected and the Home Office experiments appear to have thrown no light upon the possible causes of the explosions in America and France.

At the moment, therefore, the matter seems still to be obscure.

Cargo Shipments from Canada.

It was recently reported in Montreal that the Federal Government is to be asked to institute legislation in the next session of Parliament making it mandatory that a portion of all Canadian cargoes should be carried in Canadian ships. This measure is intended to provide protection against competition from other maritime nations.

Shipowners also have asked Britain, through the Canadian Maritime Commission, to agree to move 15 to 20 per cent. of British purchases from the Dominion in Canadian ships as it was essential they should have at least this volume of traffic to keep their ships moving steadily. Commenting on this proposal, a Canadian shipowner suggested that acceptance would be a small concession in return for Canadian help to Britain. It also was pointed out that such a move on the part of Britain would release considerable tonnage for use elsewhere and would help British shipowners to release some of their chartered foreign tonnage and thereby save dollars.

It appears that the Canadian Merchant Marine is experiencing serious difficulties, and failure to obtain assistance from Britain may compel the operators to take their ships out of service with consequent unemployment among their crews.

The Canadian Maritime Commission which was established a year ago, is expected shortly to present the shipowners' request to the British Government, and in view of the need for maintaining a strong mercantile marine throughout the Empire, it is to be hoped an agreement satisfactory to all parties will be reached.

The Panama Canal

Examination of the Sea-Level Project

Foreword

The paper which follows by James H. Stratton, M.Am.S.C.E., entitled "The Canal and the Future," is reprinted, in slightly abridged form, from the proceedings of the American Society of Civil Engineers, to whom *The Dock and Harbour Authority* is indebted for that courtesy. It is an introduction to eight other papers written by American Engineers upon the many aspects of investigation and research conducted under the powers given by the Seventy-ninth Congress which were approved in December, 1945, to determine the best means of improving inter-oceanic sea communications.

Précis of these papers have been prepared and will be printed in the December issue of this Journal. These will give the principal items of interest, together with a general idea of the methods of investigation adopted, their scope and the conclusions arrived at. The originals formed the basis of the report that was prepared, which the Governor of the Panama Canal, under the supervision of the Secretary for War, was ordered to make to Congress not later than December 31st, 1947.

It was primarily the events of World War II. that focussed attention upon the necessity for additional safeguards to be undertaken to preserve intact an adequate sea-route across the Isthmus of Central America.

It is interesting at the present time to refer to the history of the Panama Canal and to the early economic and political significance of the natural barrier of the North and South American continents joined by the Isthmus of Panama, which led to the searches made in the Middle Ages for a sea route from Europe to India and China.

The capture of Constantinople in 1453 by the Turks interrupted the ancient trade routes between Europe and the Far East, and the caravans which crossed the deserts between the Euphrates and the Indus rivers were continually attacked by robber bands, while the Mediterranean and Red Seas were infested with pirate ships which intercepted the vessels carrying merchandise destined for Italy and the Western Countries of Europe and England.

The attention of all Europe was therefore directed towards the possibility of there being in existence a sea-route Westwards to India and China, which would be free from such interferences.

Great voyages of discovery were the result between 1492 and 1530 by Spaniards, such as Columbus, Cortez, Balboa, the Portuguese Galvano, the Frenchman Cartier, and the great British circumnavigators Magellan and Drake.

It was about the year 1530 that it was finally substantiated that there was no natural waterway linking the Atlantic and Pacific Oceans, except by way of Cape Horn and the Straits of Magellan or by the possibility of a North West passage.

It became abundantly clear to the traders of the world that an Isthmian passage Westward offered great advantages over the established routes around the Cape of Good Hope, Cape Horn, or the problematic North West passage.

It appears that the honour of first proposing the project of an artificial waterway through the Isthmus of Panama belongs to a cousin of Cortez—Alvaro de Saavedra Ceron, for he prepared plans for a canal there along a route almost identical with that chosen nearly four hundred years later. This scheme, however, was not proceeded with owing to Ceron's death.

Four other routes, Darien, Panama, Nicaragua and Tehuantepec, were mentioned by Galvano, the historian and explorer, who lived in the sixteenth century.

It may cause surprise that at this early period the execution of such a feat of engineering as the construction of a canal should have been projected—the answer, of course, is that the small ships in use would have required a channel of quite small dimensions.

Charles V., King of Spain, in 1534 ordered a survey of the Chagres River, but he died before any scheme was completed.

Phillip III. of Spain also directed a survey for a Darien canal in 1616, and further Spanish surveys were made from 1771 to 1779 of the Tehuantepec and Nicaraguan routes.

Many were the projects put in hand which failed during the first half of the nineteenth century, chiefly by the Nicaraguan route, among these being a Dutch canal concession from the Nicaraguan Government, and even Louis Napoleon Bonaparte, then a prisoner of war, became interested in the subject.

The Declaration of Independence of the Central American States, together with the extension of British influence in Honduras, Nicaragua and the advance of the white settlers across the North American continent, somewhat altered the state of affairs, and it was not until 1846—when the United States acquired the States of California, Nevada, Arizona and New Mexico—that the Union began to attach greater importance to the construction of a canal and became more sensitive to rival foreign ambitions in Central America, which was particularly acute in respect to those of England in regard to Nicaragua.

Endless discussions took place and treaties were entered into between the United States on the one hand with Britain, New Granada and Nicaragua on the other as to concessions.

In 1869 an event occurred which had a decisive effect on canal affairs in Central America—the opening for traffic of the Suez Canal, constructed by Ferdinand de Lesseps.

Meanwhile the French had obtained a concession from the Colombian Government for the project of a sea-level canal on the Panama route. A corporation was formed in 1881 and soon after Lesseps commenced the operations which subsequently failed financially and by disease and death in 1889.

A new Panama Canal Company was formed in 1893, which took over all the assets of the De Lesseps Company which, after considerable conflict in the Congress of the U.S.A. as to the respective merits of a canal in Nicaragua or that in Panama, were bought by the U.S.A., who then sought to get the concession for the construction of the latter conveyed to them. Difficulties, however, arose in this respect, occasioned by European intrigues, in the background of which was Germany, with whom Colombia were also secretly negotiating as to a concession being granted to them.

In the end Panama revolted from Colombia and the U.S.A. suddenly and without further difficulty obtained all it wanted in the Isthmus.

By the Hay-Bunau-Varilla Treaty of 1904, the United States undertook to maintain the independence of the new Republic of Panama, and in return Panama granted to the U.S.A. in perpetuity the use, occupation and control of a strip of territory ten miles wide and extending three nautical miles into the sea at either end of the zone.

The cities of Panama and Colon were not embraced in the canal zone, but all canal and railway properties belonging to Panama passed to the United States, and provision was made for the use of military force and the building of fortifications for the protection of the canal.

American occupation of the Isthmus followed immediately after ratification of the treaty and a Commission was appointed to administer the canal zone and to execute the preliminary engineering and scientific investigations of the construction problems of a canal. There was considerable research and discussion into the relative merits and flexibility of the two possible forms of canal—high level lock or sea-level—before it was finally decided in 1906 to proceed with the first-mentioned type.

From the experiences of the French it was early realised that malaria and yellow fever were diseases which would have to be eradicated if success was to attend the enterprise, but in spite of

The Panama Canal—continued

great efforts deaths from yellow fever continued until a different technique for eradicating the disease-bearing mosquito was evolved and put into practice. In this respect, the drainage of swamps and the construction of modern waterworks and sewerage systems played a great part.

The present Panama Canal was finally completed in 1914 and formally opened in 1915.

Bringing the history of the Panama Canal down to the present day, it will be observed that during the greater part of the Second World War there existed a set of circumstances almost identical with those which occurred when Turkey captured Constantinople and dominated the Mediterranean, that is to say, the Allies' Mediterranean route was dominated by the Germans and Italians. Since the war, moreover, the foreign policy of Soviet Russia and her ambitions in the Middle East have focussed attention once again upon the vital necessity to the Western Powers and the U.S.A. of having an adequate sea route to the East via Central America.

The Canal and the Future

By JAMES H. STRATTON, M. A.S.C.E.*

THE long history of the search for a passage to the east, and of the early efforts to promote a canal across the Isthmus of the Americas, provide a background of romance and colour to the story of actual construction which opened in 1882 with the initiation of work on a sea-level canal at Panama by the first French Canal Company. After years of effort and mounting adversities when funds had run low, the French changed to the less costly high-level lock canal. Because the French believed that a sea-level canal ultimately would be necessary, plans for the locks were drawn to facilitate the transformation. The chapter of the story dealing with the French construction ended with financial failure and the transfer of all rights and interest in the Panama Canal to the United States.

The opening chapter of the tale of construction of the Panama Canal by the United States is one of controversy over the type of canal to be built. The records disclose that the selection of the lock canal was based on the advantages it offered in the earlier completion at a lesser cost. The issues entering into the selection have a particular interest now, in the face of the challenge presented by the "blockbuster" bomb, the guided missile, and the atomic bomb. When the present canal was planned the critical forms of attack were envisioned as naval gunfire directed against the locks and enemy forces moving overland to capture the canal intact.

The Seventy-ninth Congress expressed the temper of present concern for the security of the canal by passing Public Law No. 280, which was approved by President Harry S. Truman on December 28th, 1945, and which provides:

> "Be it enacted by the Senate and the House of Representatives of the United States of America in Congress assembled, That the Governor of the Panama Canal, under the supervision of the Secretary of War, is hereby authorised and directed to make a comprehensive review and study, with approximate estimates of costs, of the means for increasing the capacity and security of the Panama Canal to meet future needs of interoceanic commerce and national defence, including re-study of the construction of additional facilities for the Panama Canal authorised by the Act approved August 11th, 1939 (53 Stat. 1409). He shall also make such study without drafting plans or sketches as he may deem desirable to permit him to determine whether a canal or canals at other locations, including consideration of any new means of transporting ships across land, may be more useful to meet the future needs of interoceanic commerce or national defence than can the present canal with improvements. He shall report thereon to the Congress, through the Secretary of War and the President, not later than December 31st, 1947."

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Traffic History of the Canal

The volume of traffic through the Panama Canal has steadily increased since it was opened, despite the setbacks of wars and world depressions. With uninterrupted world prosperity a further growth of commercial traffic may be expected generally as predicted by Roland L. Kramer of the University of Pennsylvania at Philadelphia in recent studies which were made for use in the investigations under Public Law No. 280.

Before the war the "tolls-free" traffic, consisting largely of vessels owned and operated by the United States, accounted for about 15% of the entire traffic. In 1945, the peak war traffic year, this rose to 78%. Since the canal was opened in 1914, it has transited 198,000 ships, of which 142,000 were tolls-paying commercial craft.

War traffic through the canal in the period from 1941 to 1945 totalled nearly 17,000 transits. Had there been no canal during this period, it is estimated that the increased ship-operating costs and the cost of additional shipping and escort craft that would have been required to preserve the schedules made possible by the canal would have exceeded $1,500,000,000.

From its contribution in peace and war, it is clear that the future of the canal is the future of ships, provided the canal can be made safe against destruction.

The Panama Canal as it is To-day

The canal as a waterway has changed little since it was placed in service, except for the addition of the Madden Dam on the Chagres River in 1935 for the benefit of water supply for lockages, flood control, and power generation.

A plan and profile of the existing canal are shown in Figs. 1 and 2. It is estimated that by about 1960 the capacity of the canal will be inadequate to accommodate traffic without inflicting undesirable delays on peak traffic days. The delays thereafter will become more serious with the further growth of traffic unless additional capacity is provided.

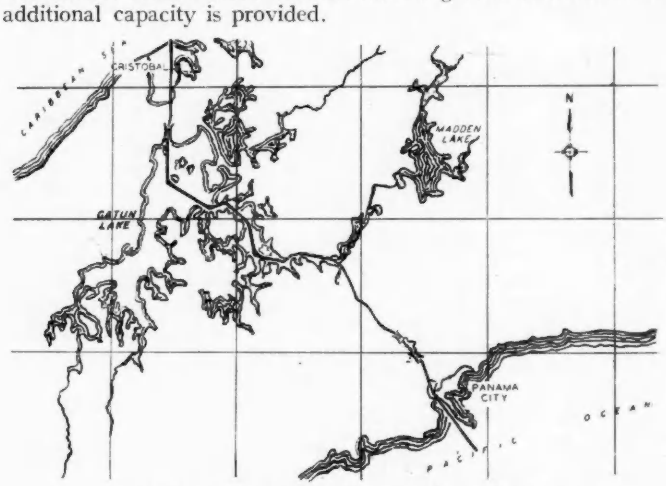


Fig. 1.—General Plan, Panama Canal Lock.

The restricting effect of the small locks (width 110 ft., length 1,000 ft.) of the present canal on the design of Navy ships became intolerable with the approach of war and, in 1939, Congress directed the construction of a third set of locks, 140 ft. wide and 1,200 ft. long, which it was then thought would be adequate for all future needs. The new locks were designed to resist attack by the largest aerial bomb then known to exist. Construction was suspended early in 1942, when it became apparent that the new locks could not be completed before the end of the war because of conflicting demands for men and materials. At that time, excavation for the Gatun and Miraflores Third Locks had been substantially completed, but excavation for the third lock at Pedro Miguel and work on actual lock construction had not commenced. Of the authorised expenditure of $277,000,000, approximately $75,250,000 was spent.

The Panama Canal—continued

Limitations of Present Canal and Future Traffic Needs

The present locks are expected to be adequate dimensionally for all commercial shipping for the remainder of the twentieth century except for ships of the "Queen" class which do not, and ordinarily would not, use the sea route through the canal. The limiting effect of lock size on the passage of naval ships is expected to become even more stringent in the future than it is at present.

The commercial canal tonnage predictions of Professor Kramer were transformed into expected future ship transits by the engineering staff employed on the studies by taking into account the trends in ship sizes, the expected character of future cargoes, and the expected proportion of transits under full and partial load, and in ballast, as evidenced by past experience. Thus, the transportation of 87,770,000 long tons of commercial cargo estimated by Professor Kramer for the year 2000 would require 13,078 ship transits. In the year 2000, commercial and tolls-free traffic would average 46 transits daily. On peak traffic days 69 transits could be expected. The year 2000 was selected to establish the period during which future traffic needs must be met if any major construction or reconstruction is undertaken.

By locking small ships in tandem and taking into account the sizes of ships estimated for the future, the peak load of 69 ships could be realised with 46 lockages using the present locks. With locks 200 ft. wide and 1,500 ft. long (which is the size now recommended by the Navy to meet its future needs), it would take 29 lockages to effect the passage of the 69 ships.

How Secure is the Present Canal?

Although no one can say what course World War II would have taken had the Japanese followed up Pearl Harbour (Hawaii) with an attack on the Panama Canal, it is now clear that the locks could have been destroyed and the canal emptied into the sea had an attack been made and the defences penetrated. The development of larger bombs and new weapons of both conventional and atomic types since Pearl Harbour leaves no doubt as to the vulnerability of the canal to enemy attack and sabotage. If the needs of national defence are to be met, the canal must be made secure against attack and sabotage.

The penetration of canal defences by rockets, guided missiles, or robot planes loaded with powerful conventional or atomic explosives launched from the air, from ship or submarine at sea, or from a land base, must be accepted as a possibility. A single sneak attack could destroy the lock gates of the present canal and drain Gatun Lake into the sea. If the needs of national defence are to be met, steps must be taken to make the canal secure.

Third Locks Project in Review

A new third set of locks to accommodate large naval vessels, no matter how strongly constructed to resist attack and sabotage, would not add to the over-all security of the canal because the existing locks and impounding dams cannot be strengthened sufficiently to give them equivalent protection. If any lock is breached and Miraflores Lake or Gatun Lake is lost, the canal would be closed for months or even years for repairs and for the restoration of the lost lake. In the light of the threat of present weapons, the Third Locks Project would provide for only the peacetime needs of commerce, which, if that were the sole consideration, could be met by other means at a considerably lesser cost.

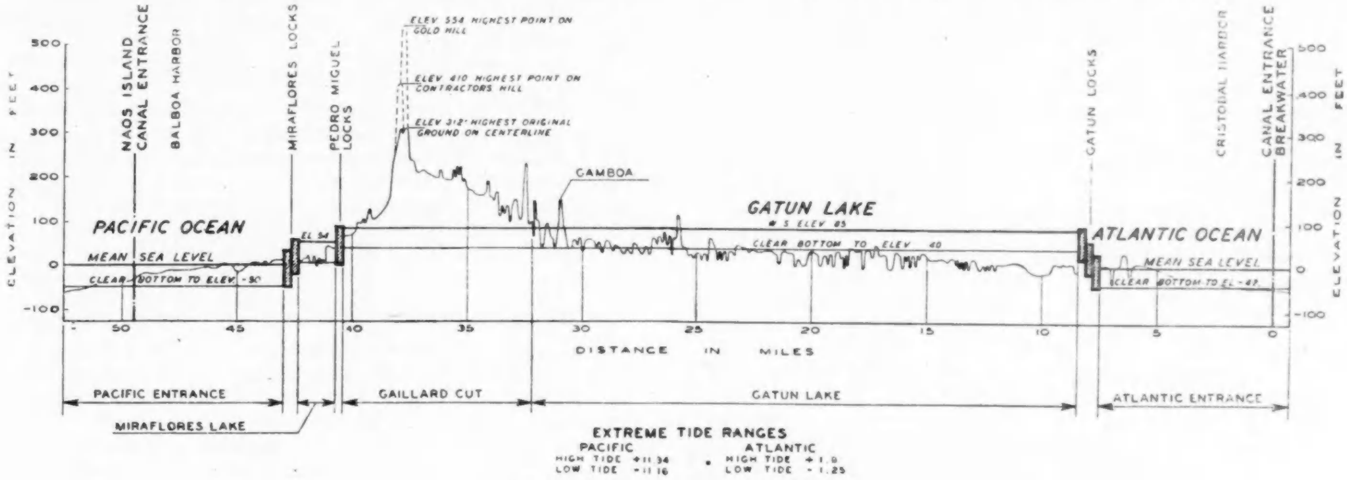


Fig. 2.—Profile Panama Lock Canal.

Improvements in the Interests of Commerce Only

The requirements of interoceanic commerce for the remainder of the twentieth century could be met by the elimination of lay-up of the locks for overhaul and by overcoming fog interference with traffic. The first of these is the more restrictive on the capacity of the canal, reducing it from 58 ships to 36 ships per day. Each lock is now overhauled every four years, repairs being undertaken every two years, alternating between the Atlantic (Gatun Locks) and the Pacific Locks (Miraflores Locks and Pedro Miguel Locks). While one lane of locks is under repair, the adjacent lane of locks is kept open to traffic; thus the availability of only a single operating lane of locks at one end or the other of the canal, during the period of overhaul (about 4 months), establishes the dependable canal capacity. Repairs are made in the dry season when there is no fog, and the canal is then operated 24 hours daily instead of 16 hours per day, as is the case in the rainy season when fogs are of relatively frequent occurrence after midnight. Round-the-clock operation and careful scheduling of transits make it possible to hold the reduction in capacity due to overhaul of the locks to less than one half of the normal operating capactiy of the canal.

Repairs to the lock gates and their mountings, and to the operating machinery and the filling culvert valves and fittings, are undertaken with the lock chamber dewatered, as are the cleaning and painting of all underwater metal parts. The lay-up and dewatering of the locks for repairs could be eliminated by providing new gate mountings of a special type and new type lock gates having buoyancy chambers to float them out and into position, thus effecting replacements in the matter of a few hours. Repairs to gates would be made in dry dock. Alterations to avoid lay-up of the locks for overhaul and certain channel improvements would raise the dependable capacity of the canal to 65 ships per day.

Fogs of the ground-radiation type, which occur in Gaillard Cut in the rainy season (generally from May through December between midnight and daybreak), require closing the canal to traffic during these hours. Various methods of dispersing fog have been employed for the clearing of airfields, but present indications are that they would be too expensive for dispersing fogs in the canal. Developments in electronic aids to navigation offer the prospect

The Panama Canal—*continued*

of a cheap and fully reliable method of passing ships through a fogbound restricted channel. By the time traffic demands require 24-hour operation of the canal, these devices will be considerably improved and undoubtedly can then be adapted to the canal needs to increase its dependable capacity to 70 ships, which would be adequate for the remainder of the twentieth century.

The cost of the various improvements to extend the life of the present canal in the interests of commerce only would be $129,983,000. Any expenditure in excess of this amount can be justified only by the requirements of national defence.

A Modernised Panama Lock Canal

A reconstructed lock canal with all Pacific locks grouped at Miraflores to provide full lift to summit level would offer the greatest economy of construction and would create an anchorage area at the head of the Pacific locks that would facilitate the dispatch of vessels through Gaillard Cut. A similar arrangement of the Pacific locks was proposed by Adolpe Godín de Lépinay in 1879 and was advocated by the late W. L. Sibert, M. A.S.C.E., in 1908. Colonel Sibert's suggestion was rejected because of the advanced state of the planning and construction, and doubts as to the foundations for three-lift locks at Miraflores, the site he proposed for the Pacific locks. The Miraflores foundations have since been found satisfactory for locks with full lift to summit level.

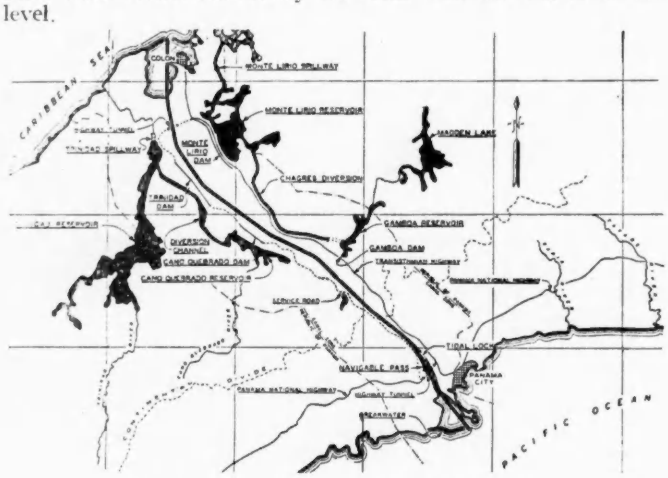


Fig. 3.—Plan of Panama Sea-Level Canal.

Two dispersed locks would be provided at Gatun and Miraflores with two lifts to attain summit level instead of three as at present. Chambers would be 200 ft. wide, 1,500 ft. long, and 50 ft. over the sill. By raising Gatun Lake to El. 92, from its present maximum El. 87, additional storage for lockage water would be provided. This augmented water supply would not meet lockage demands until the year 2000 and supplementary pumping from the sea eventually would be necessary.

The dispersion of the locks and their armouring with concrete and steel to protect the lock machinery and the culverts would provide them with the highest practicable degree of protection. The lock gates do not lend themselves to protective treatment, except against the lightest type of aerial bomb. However, multiple sets of gates, well dispersed, would increase the difficulties of dissipating Gatun Lake. Certain of the gates would be housed in protected recesses when not in use. The closure dams adjoining the locks at Gatun and Miraflores would be of massive earth construction. The Gatun Dam spillway would be channelled in the rock abutment for maximum protection.

The cost of such a fully modernised lock canal at Panama would be $2,307,686,000.

Security considerations restrict public evaluation of the protective designs in relation to the various weapons that could be employed in an attack. It can be stated, however, that the lock canal cannot be made resistant either to atomic bombs or to modern conventional weapons. At best the protection that could be provided for a lock canal would only increase the difficulties rendering it useless. The modernised locks could be breached by a determined enemy and the canal could thus be closed to traffic for the period required for reconstruction and for the capture of the tributary runoff to restore the summit lake, which might require as much as four years. The extent of damage and the length of the period of traffic interruption would depend on the nature of the weapon employed and the intensity of attack. Radioactive contamination would make repairs to the locks difficult if not impossible. The lock type of canal, no matter how strongly constructed, would not increase security to meet the needs of national defence.

Sea-Level Canal Possibilities

The various sea-level canal route possibilities were narrowed down to eight. The Panama route (Fig. 3) is the least costly and has the additional advantages of an operating and administrative establishment and of defences already in place. These installations would have to be duplicated at any other sea-level route. The plan of development for a sea-level canal at the several routes was similar to that for a sea-level canal at Panama as described in the next section.

Plan of Development of a Sea-Level Canal at Panama

There are numerous possibilities for a sea-level canal in the Canal Zone and in the immediate vicinity. The route that would cost the least and have other outstanding advantages is designated the Panama sea-level conversion route, since it follows generally the alignment of the present lock canal. The distinctive feature of a canal on the conversion route is that its construction would involve lowering the present canal to sea level, whereas this would not be the case if either the Panama parallel or one of the Chorrera canals were constructed.

In the Gatun Lake area, the Panama parallel canal would be separated from the existing lock canal by a barrier dam constructed from spoil material from the excavation for the new canal. From Gamboa south to Balboa Harbour it would follow an alignment separated from the present canal. Thus, the Panama parallel sea-level canal would be completely independent of the lock canal and, as would be the case if the Chorrera-Lagarto canal were constructed, both the existing canal and the new canal could be maintained and operated if this were thought necessary. There are no reasons arising either from capacity or security considerations that would justify the additional $800,000,000 for the Panama parallel sea-level canal. Any one of the Chorrera routes would be even more costly than the Panama parallel route and would be no more justified.

The alignment of the present canal and that of the proposed Panama sea-level conversion canal would be sufficiently separated at several beaches to make it possible to construct a substantial part of the latter in the dry, thus avoiding interference with canal operations. The alignment improvements introduced in the conversion route would shorten the canal by 5.2 miles

The Sea-Level Canal Channel

The channel of the Gaillard Cut in the present canal is deficient in width for two-way traffic involving large and unwieldy ships, and single directional transiting arrangements for such ships are in effect to avoid their encountering other ships in the cut. The delays and the inconveniences resulting from the special handling of this type of traffic have not thus far been seriously objectionable, but they would be with the further growth of traffc, and this fact was borne in mind in designing the sea-level canal.

Currents up to 4.5 knots would be induced in a sea-level canal at Panama without tidal control by the Pacific tides, which have a range up to 20 ft. Atlantic tides have a maximum range of 2 ft. and would cause currents of 0.5 knot if the Pacific tidal effects were eliminated by control works at the Pacific entrance.

In establishing the design of the sea-level channel, a world-wide survey was made of comparable channels and canals. Several waterways having characteristics similar to the Panama sea-level canal, including the sea-level canals at Suez (Egypt) and Cape

The Panama Canal—continued

Cod (Massachusetts), were visited by members of the staff engaged on the studies. In addition, model investigations were made for The Panama Canal by the United States Navy at the David Taylor Model Basin at Carderock, Md., to determine, for both the lock and sea-level canals, the width, depth, and alignment requirements. For the sea-level channel determinations, various widths and depths of channel using both straight and bend channel sections were tested with currents up to 5 knots. Self-propelled, remote-controlled models of typical ships that transit the canal, operating at a wide range of ship speeds, were employed in testing the channels.

Two-way traffic of most of the ships that would be expected to transit the canal could be accommodated in a channel designed for the safe meeting of a standard "Liberty" ship by the largest naval craft or the largest commercial craft now afloat. The meeting and passing of the largest present-day naval craft and commercial vessels would be unusual and could be avoided by special transiting arrangements similar to those now in effect. Similar arrangements could be made for the transit of the largest ships expected to be built during the twentieth century.

In spite of considerations leading to the recommendation of the Governor of The Panama Canal (J. C. Mehaffey) that tidal currents in the canal be regulated, the channel was designed for safe navigation in currents up to 4.5 knots. This is made necessary by reason of the fact that the structures for the regulation of currents in the canal could be irreparably damaged by bombing and would have to be cleared from the channel. As the result of the studies, it was concluded that all shipping could safely transit the proposed Panama sea-level canal at any condition of current that would obtain up to the maximum of 4.5 knots; only an occasional unwieldy or low powered ship would be held for mean tide to avoid encountering high currents. Tug assistance could be provided such ships to avoid delays and to provide increased safety of transit when thought necessary.

The channel standards adopted as the result of the model studies and other investigations are given in Table 1, as are the controlling standards of the present canal.

Table 1—Comparison of Channel Dimensions

Description (1)	Depth (ft.) (2)	Width (at a depth of 40ft.) (ft.) (3)	Cross-sectional area (minimum) (ft.) (4)	Minimum sight distance (miles) (5)	Maximum deflection angle (degs.) (6)	Angularity Total (degs.) (7)	Angularity Per mile of Canal length (8)
Present Panama Lock Canal ...	42	300	13,860	0.6	67*	598	11.7
Modernised Panama Lock Canal	55	500	28,400	0.6	67*	642	12.5
Panama Sea-level Canal ...	60	600	36,800	1.5	26	117	2.5

* This applies to Gatun Lake: in Gaillard Cut the maximum angle is 30°.

The survey of the world waterways disclosed that channel depths are generally considered inadequate and that better ship controllability would result with more water under the keel. The Carderock tests fully confirmed the latter conclusion. The lesser depth proposed for the projected lock canal results from the lesser ship speed (8 knots) that would be prescribed in the restricted channel of a lock canal. A ship speed of 10 knots was used to establish the depth of the projected sea-level canal. The width of channel is referenced in each case to the 40-ft. depth below the low water surface, which approximates the draft of the large vessels of the future. This depth also establishes the point of revolution in setting bank slopes, thus minimising variations in the surface width of the channel arising from differences in the slopes, which are fixed by the character of the bank materials.

The results of the model tests were interpreted with the assistance of the United States Navy and the pilots of the Panama Canal and of the Cape Cod Canal. The advice and counsel of the Panama Canal pilots on all problems dealing with navigation were invaluable in the conduct of the studies.

Sea-Level Canal Model Investigations

George B. Pillsbury, M. A.S.C.E., has developed a procedure for computing currents in sea-level canals which, when applied to the proposed Panama sea-level canal, yielded results that were closely confirmed by Boris A. Bakhmeteff, Hon. M. A.S.C.E., by independent computations and by model investigations. A model of the proposed sea-level canal at an undistorted scale of 1 : 100 provided methods of accurately determining conditions of flow in the uncontrolled waterway at all ranges of tide, of establishing the design, and of testing the tidal-regulating structures. The maximum velocities in the unregulated canal 60-ft. deep and 600-ft. wide at 40-ft. depth at various Pacific tides as computed and as observed in the sea-level canal model are given in Table 2.

Table 2—Velocities in an Unregulated Sea-level Canal

Table 3—Schedule of Navigable Pass Opening in a Regulated Sea-level Canal

Tidal range (ft.) (1)	Percentage of time tides are exceeded (2)	Maximum Velocities Uncontrolled Channel Observed (knots) (3)	Maximum Velocities Uncontrolled Channel Computed (knots) (4)	Permissible current velocity (knots) (1)	Hours per day that the Navigable Pass would be open for Tidal Ranges of: 6 ft. (2)	10 ft. (3)	13 ft. (4)	16 ft. (5)	20 ft. (6)
6	99	2.1	2.1	1	8.5	4.5	3.5	2.9	2.3
10	80	2.7	3.0	2	20.2	9,7	7.2	6.0	4.9
13	50	3.3	3.5	3	24.0	24.0	14.9	11.6	9.1
16	20	3.8	4.0	4	24.0	24.0	24.0	24.0	19.8
20	2	4.4	4.5	4.5	24.0	24 0	24.0	24.0	24.0

Tidal Regulation for the Sea-Level Canal

The majority (eight out of thirteen) of the Consulting Board appointed by President Theodore Roosevelt in 1905 to consider various plans for a canal at Panama prepared by the Isthmian

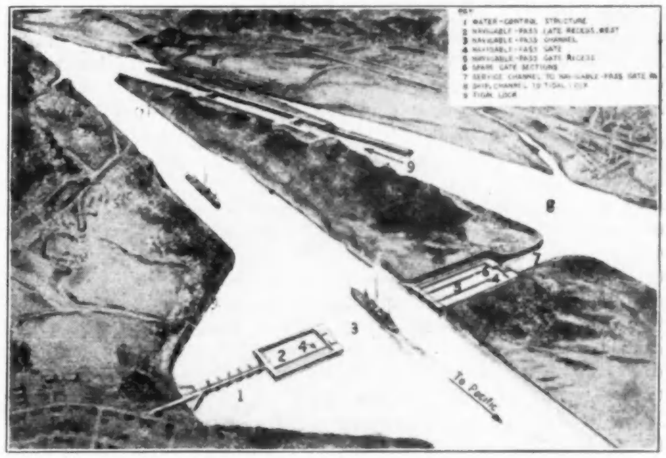


Fig. 4.—Tidal-regulating structures showing the Navigable Pass open.

Canal Commission in voting for a sea-level canal stated with respect to tidal control:

> "The plan [sea-level] proposed by the Board for the Isthmian transit will have twin tidal locks near the Pacific terminus, which if disabled, one or both [by enemy attack or sabotage] would still be usable (after removal of wreckage) for a part of each day (the period of spring tides) in each lunar month, and probably throughout the whole twenty-four hours the remainder of the lunar month (neap tides)."

If tidal regulation were to be omitted, occasional ships would need to be held at the canal entrances for a favourable tide, or tug assistance would need to be provided for their transit. The Carderock tests and experience in other waterways are conclusive in demonstrating that relatively few modern ships would require special transiting arrangements; the majority, having reliable power and rudder control, would be capable of transiting the Panama sea-level canal safely in currents up to 4.5 knots. Nevertheless, because of the serious consequence of collisions with the canal banks (which would be largely of rock), it was decided to provide tidal regulation for added safety to shipping. The loss of tidal regulation as the result of bombing of the regulating

The Panama Canal—continued

structures is a definite possibility, and is accepted as a condition of operation during war-time.

If the control of the Pacific tides were to be provided by tidal locks and a barrier dam and these were to be damaged by the enemy, then the canal would either have to be closed for repairs or the structures cleared from the channel to permit use of the canal as an open waterway. In the latter case, there would be an abrupt transition from slack-water navigation to navigation in currents ranging up to 4.5 knots. Prolonged closure of the canal

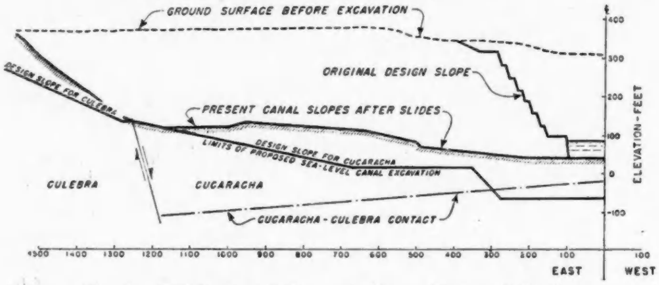


Fig. 5.—Original and Present Slope Design Criterion (Old Canal Station, 1785 + 00).

for repairs would be intolerable, which suggests that the tidal-regulating structures should be planned to permit the rapid clearing of a "channel-way" in case they are damaged. This could be done by excavating an auxiliary channel which would normally be closed by an earth barrier dam that could be rapidly blasted to clear the channel when the locks were rendered unusable by bombing. Obviously, an abrupt change from navigating a canal with currents completely regulated to one with currents of the order listed in Table 2 would be undesirable and this led to an investigation of other methods for providing tidal regulation.

A sudden transition from slack-water navigation to navigation in a completely unregulated waterway in the event of damage to the tidal lock could be avoided by supplementing the tidal lock with a navigable pass through which ships, as a matter of routine operation, could pass at any selected stage of the tide. The pass would have retractable gates (Fig. 4), which could be opened rapidly to permit navigation without the need for lockage under any set of conditions of current desired, from 0.5 knot (Atlantic tidal currents) to the maximum. The gates of the pass would be of steel construction, and in the event of their damage could be readily removed from the channel. Table 3 shows the hours daily that the pass would be open if the tidal currents were controlled to 1, 2, 3, 4 and 4.5 knots. These data were determined by computations and confirmed by sea-level model tests.

It is probable that, when the sea-level canal is first placed in operation, the navigable pass would be opened only at mean tidal stages to limit channel currents to low values. As operating experience is gained, the periods of opening of the pass gates would be extended to permit higher currents in the canal. Tentatively, currents in the canal would be limited to 2 knots; this value was selected after a survey of operating experience in other waterways and as the result of the United States Navy tests at Carderock. On this schedule for the opening of the pass gates, the canal would operate 32% of the total time as an open waterway.

A gated water-control structure would complement the navigable pass to assist in adjusting the water surface in the canal to extend the period of opening of the pass at each mean stage cycle of Pacific tide (Fig. 4). The gates of the water-control structure would be open only when the pass is closed to traffic. The tidal lock could accept ships at all stages of the tide and would be the sole avenue of transit when the pass is closed.

The sea-level canal model was an invaluable aid in planning the location, the arrangement, and the hydraulic features of the separate elements of the tidal regulating works and in developing the operating schedule of the navigable pass.

One important phase of the model study was that of relating the friction factor of the model to the friction factor of the prototype channel. This was accomplished by actual flow tests in Gaillard Cut with the culverts in the Pedro Miguel Locks wide open and discharging 22,000 cu. ft. per sec.

Slides in the Panama Sea-Level Canal

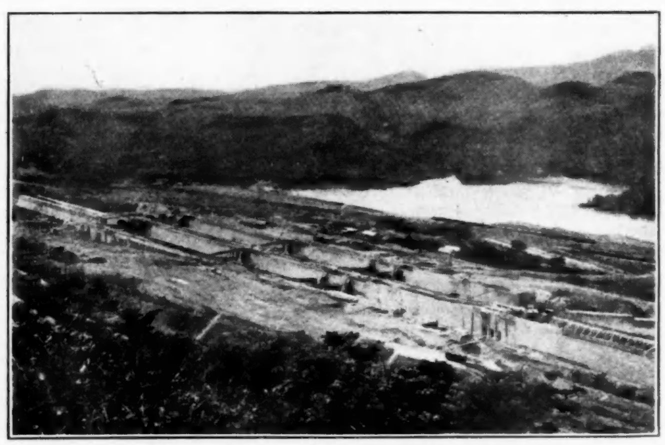
A study of the major slides experienced in the period of the canal construction was of great assistance in comprehending the character and probable behaviour of the materials to be encountered in any new excavation. These slides, as was stated by the late George W. Goethals, M. A.S.C.E., in 1916, were the result of an attempt to fix uniform slopes throughout the length of the canal, regardless of height of cut and character of materials. General Goethals stated after the completion of the canal:

> ". . . With the geological formation changing so frequently and so suddenly both in the direction of the cut and up and down there is no possibility of any uniformity in slopes. No uniformity of slopes could be maintained . . ."

For a time after the first major construction slide took place, it was thought that the slopes would eventually stabilise themselves and that the volume of material ultimately to be removed would be less if the slides were allowed to run their course. This proved not to be the result; moreover, the pressing need for opening the canal led to flattening certain of the slopes to prevent further blockage of the channel. Fig. 5 represents for a specific location the design slopes now proposed for the Cucaracha formation, and shows the slope selected in the initial construction and the actual slope attained in this formation after failure.

Consideration was given in the design of slopes to dynamic loading that could be induced by bombing. A study of the resistances of the materials to dynamic (transient) loadings was made at Harvard University in Cambridge, Mass., under an investigational contract. The studies of the resistances of the materials to transient loadings and the tests of their residual strengths after failure will be of particular interest to engineers. The transient-load testing apparatus developed at Harvard University offers promise as a valuable tool in developing fuller understanding of the behaviour of soils and rocks under stress.

No allowance was made in the designs for slide failures resulting from dynamic loadings since it is unlikely that slides induced by the atomic bomb would fully block the canal. However, debris from cratering resulting from atomic bombing could block the channel, but widening the channel to overcome this possibility would require such a large amount of excavation that the risk of closure was accepted, since in any case the blocking materials, even though radioactive, could be removed in a few weeks at the outside. None of the conventional weapons could induce slides or throw up craters that would block the channel of either a sea-level or an improved lock canal.



Pedro Miguel Locks: View from Hill on East Bank.

Panama Canal—continued

Conversion to Sea Level

Studies of the conversion of the Panama lock canal to a canal at sea level prior to that under Public Law No. 280 contemplated the lowering of Gatun Lake in stages. One study planned for the lowering of Gatun Lake in seven stages. Philippe Bunau-Varilla, the French engineer who negotiated the sale of the canal holdings of the French to the United States, urged the consulting board appointed by President Theodore Roosevelt to provide deep upper sills at each of the locks to facilitate the later conversion of the lock canal by stages to a sea-level canal. His plan was rejected on the grounds that major modification of the proposed twin locks would be inescapable in any case and that a third set of locks would be needed in order that two lanes of locks would be available for traffic at all times during the conversion period.

There is no question that the conversion of the existing locks would involve considerable risk both to shipping and to the integrity of the canal with only one lock lane available and that no plan should be accepted that would have less than two lanes of locks available to shipping throughout the period of conversion. If the lowering of the summit lakes to sea level by stages is undertaken, new special twin conversion locks of minimum construction with lift to El. 53 are preferable to previously considered arrangements involving the progressive alteration of the existing locks for each stage of lowering using a new third set of locks with full lift to El. 85 to insure two lanes being available at all times. The proposed special twin conversion locks would be placed in operation and the existing locks abandoned when the first stage of lowering of Gatun Lake from El. 85 to El. 53 is accomplished—or after the completion of excavation to prepare the channel to carry traffic with the summit lake at El. 53. The next stage of lowering of the canal would be to El. 22, and finally the lowering would reach sea level; in each case excavation to provide necessary depth of channel would precede lake lowering. The twin conversion locks have low upper sills to accommodate traffic at all stages of canal elevation from El. 53 to El. 0. This plan and others considered for the stage lowering of the summit lakes were finally abandoned because of the attractiveness and the cheapness of the plan for the single-stage lowering of the summit lakes.

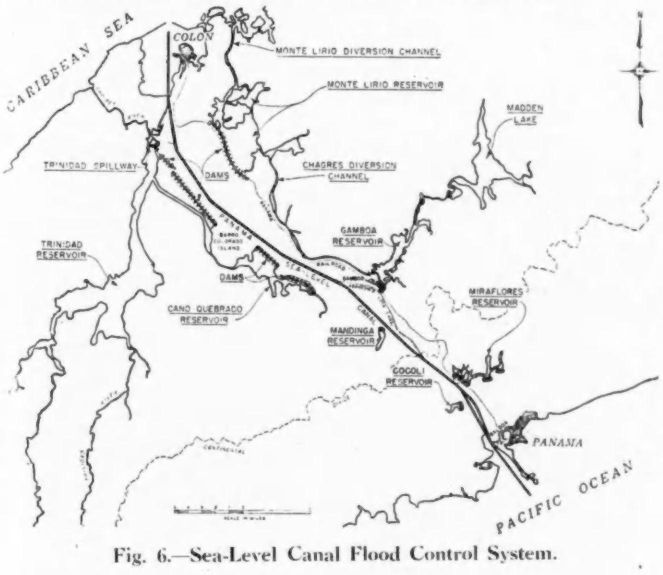


Fig. 6.—Sea-Level Canal Flood Control System.

The conversion of the lock canal to sea level by lowering the summit lakes in a single operation would require dredges capable of excavating to a depth of 145-ft. For this purpose a cutterhead-type suction dredge would be used to excavate the softer materials and a chain-bucket type dredge, to excavate blasted hard materials. The suction dredge would have a booster pump mounted in a well on its ladder to effect the lift of the materials. The chain-bucket dredge would be similar to the conventional type used in mining of gold and tin, except that the 2 cu. yd. buckets proposed are considerably larger than those ordinarily used to depths of the order proposed. Dredges of the chain-bucket type worked successfully to depths of 128-ft. in the mining of gold; however, the buckets generally have not exceeded ½ cu. yd. The Board of Consulting Engineers and others advising Governor Mehaffey on the current studies were unanimous in their opinion that deep dredging to effect single-stage conversion would be practical and economical.

In the deep-dredging method of conversion (single-stage lowering of the lakes), there would be a net saving of $130,000,000, principally through the omission of conversion locks. The actual lowering of the water surface to effect the conversion to sea level would be accomplished by progressive demolition of natural rock plugs and the removal of a temporary steel dam left to retain the Gatun and Miraflores lakes. The conversion could be effected with a traffic interruption of only seven days.

Alignment improvements would permit a large part of the excavation to be done in the dry (750,000,000 cu. yd.) at a large saving in cost. There would be 318,000,000 cu. yd. of wet excavation classified as follows:—

Cubic Yards		Classification
9,400,000	....	Hard rock.
97,700,000	... ...	Medium rock.
46,600,000	... ...	Soft rock.
165,000,000	... ...	Sands, gravels and clays.

Of this amount, approximately 132,000,000 cu. yd. would lie between depths of 85-ft. and 145-ft. below maximum water surface and would require the employment of special dredges

Flood Control in the Sea-Level Canal

Unless tributary inflows into the sea-level canal are controlled, there would be interruptions in service and occasional hazard to navigate. The most essential control is that of the Chagres River, the major tributary; this would be accomplished by a dam constructed at Gamboa to operate in tandem with the upstream Madden Dam. The regulated outflows of the Chagres River would be excluded from the canal by diverting them to the sea through a short tunnel and a channel as shown in Fig. 6. The flows from all other tributaries on the east side of the canal north of the Chagres River would be intercepted and conducted to the Caribbean (Las Minas Bay) by a system of dams and the channels which convey the regulated flows from the Chagres. This control system, termed the "East Diversion," would have sufficient capacity to divert all floods up to 25% larger than the maximum flood of record. Emergency outlets into the canal are provided for larger floods.

"The West Diversion" system of dams, reservoirs, and channels would exclude the entry of flows into the canal from all the important tributaries west of the canal and north of the Continental Divide by diverting them into the channel of the lower Chagres through an outlet and spillway west of the present Gatun Dam.

South of the Continental Divide there are a number of small tributaries (maximum drainage area 13 sq. miles) which could contribute undesirable flood inflows if not regulated. Diversion being impracticable because of the topography and the developments in the area, a system of regulating reservoirs as shown in Fig. 6 would be provided.

The proposed combination of reservoirs and diversion systems would control 1,240 sq. miles of the total of 1,358 sq. miles of area tributary to the canal between Gatun and Balboa. Of the uncontrolled drainage areas, the largest would not exceed 6 sq. miles. The flood contribution from the uncontrolled areas would neither inconvenience navigation nor be a hazard to ships in transit.

The flood-control dams would be constructed from excavation spoil except where the length of haul would make local borrow cheaper. The east and west diversion dams would be low structures of earth and rock spoil mounted on broad platforms of fill constructed of excavation spoil placed in the wet, thus insuring very conservative loadings where the foundations are muck and soft clays. The earth and rock flood-control structures would be readily repairable in case of damage by enemy bombing.

Panama Canal—continued

Sea-Level Canal Construction Plan

It is planned to complete the construction of the sea-level canal in 10 years. The construction plan adopted in the report to Congress provides for the use of large shovels and draglines (25 yd. or larger) dumping into scows for the haul of dry excavation, totalling 750,000,000 cu. yd., to disposal areas in Gatun Lake. Wet excavation to customary depths would be performed by conventional dredges, but for depths beyond the range of this equipment, special dredges capable of excavating to 145-ft. below the water surface would be required. Vehicular tunnels under the canal would be constructed at the Atlantic and Pacific ends to aid construction and for cross-channel access thereafter. Total cost of the sea-level canal is estimated at $2,483,000,000.

Capacity of the Panama Sea-Level Canal

The capacity requirements for the year 2000 on peak days would be 69 transits. For planning purposes, ship speeds in the sea-level canal were established at 12 knots ground speed travelling with the current and 8 knots against current. A conservative spacing of 1.5 nautical miles between ships was selected after a survey of the practices of other waterways and with the advice of Panama Canal pilots. The daily capacity of the sea-level canal, based on a 16-hour operating schedule for both the tidal lock and the navigable pass, and assuming the latter would be opened to limit currents up to 2 knots, would be 116 transits daily. The average transit time in the sea-level canal would be 4.5 hours. This compares with an average transit time of 8 hours in the present canal.

Security of the Panama Sea-Level Canal

The tidal-regulating and flood-control structures of the sea-level canal would not be essential to its safe operation; hence their damage or destruction would result only in temporary interruptions to traffic. An adequate flood-warning system would insure against ships being caught in the channel at the time of incidence of large flood inflows in the event of damage to the flood-control structures. The flood-impounding structures being of earth and rock construction would be highly resistant to bombing but, because they are not absolutely vital to the operation of the sea-level canal, the costly treatment necessary to make them resistant to the atomic bomb would not be warranted. If damaged by bombing, the flood-control structures could be readily repaired and restored to service.

The tidal-regulating works could not be made resistant to modern weapons. Their loss by enemy action would be the loss of convenience to shipping and a lessening of safety in transit, particularly for ships with low power and poor rudder control. The risks in transit of such ships could be avoided by having them await a favourable tide or by providing them with tug assistance. The tidal lock, if heavily damaged, would be a mass of debris that would be difficult to remove. If the tidal lock were contaminated by radioactive particles as the result of atomic bombing, it would probably have to be abandoned. In either case the navigable pass could be used for transiting traffic. If the pass gates were damaged, they could be removed from the channel in a few days to a few weeks, depending on the extent of damage to the channel-way. Thereafter, the canal would operate as an open waterway.

None of the conventional weapons known to-day could induce slides or blockage of the channel by cratering that would close the sea-level canal to traffic. However, the canal could be blocked for limited periods as the result of an atomic weapon attack. It is estimated that a crater blockage of the canal would require a few weeks, at the most, for the clearing of a traffic lane, even though radioactive contamination would delay the initiation of the removal work. The shielding of the dredges and other excavation equipment to protect crews would be required, and this is considered practicable.

An attack employing conventional or atomic weapons could do great damage to the housing and administrative facilities of the canal and could result in great loss of life but, disastrous as the loss of life would be, the operation of the sea-level canal could go on uninterrupted since ships in an emergency could transit the canal through the navigable pass under the pilotage of their masters.

(To be continued)

Book Reviews

Baugrand and Banwerk (Foundation Soils and Foundations), by Koegler and Scheidig. Publishers, Wilhelm Ernst and Sohn, Berlin, 1948. 268 p.p., 298 illustrations, paper covers. Price 18 German marks.

The fifth edition of this book has been revised and brought up to date by Dr. Alfred Scheidig. It was first published in 1938 and immediately attracted attention on account of the thorough manner in which the authors analysed the practical application of Soil Mechanic Principles.

The present edition has several additional features, notably in the sections dealing with the Settlement of Buildings supported on raft foundations, and a more extensive treatment in the chapter on Ground Frost and its effect on shallow and unprotected foundations. Fortunately this latter phenomenon is not of serious import in this country and is amply covered by local Bye-laws and Regulations.

An additional chapter on the legal responsibility for the failure of foundations strikes an ominous note. The questions discussed are: Who should shoulder the liability for a correct evaluation of the bearing capacity of sub-foundation soil? Who should pay for the examination (research) of the ground at site?—and who should bear the risk of foundation stability on bad ground? These are important questions, but unfortunately propounded in the first instance by engineers themselves. It is some years since Professor Terzaghi, in an address to the Institution of Civil Engineers, pointed out, on the assumption that since he had developed a scientific theory of clay ground consolidation, which had been everywhere accepted by the profession, the liability for foundation failure was now the designer's risk; in other words, Foundation Engineers should be called upon to guarantee the stability of the foundations designed, or constructed, by them. This, in practice, assumes that the engineer has not only verified and appreciated, or assessed, the behaviour characteristics of the sub-strata, but also the elemental hazards.

It is somewhat unfortunate that the scientific development of Soil Mechanics, which is still in its swaddling clothes, should already be harassed by legal speculations, especially those having their origins in academic circles. Although it is incumbent upon designers to envisage future progress, and consequential liability, it is much more important, particularly in the present state of the world, to concentrate effort upon practical economic achievement.

The subject matter is highly technical and covers a number of difficult problems, and to the Foundation Engineer, versed in the German language, would be an invaluable guide to up-to-date analytical method.

Brown's Nautical Almanac for 1949," 744 pp., with numerous tables and charts, published by Brown, Son & Ferguson, Ltd., Glasgow; price 6s. 6d.

The present issue is the 72nd annual publication of this useful Almanac, and the lay-out is similar to that of previous years. The book is sectioned off in parts numbered 1 to 7, as follows:—1, Astronomical data as used daily by navigators, and an explanation of its application. 2, Nautical Tables and Methods. 3, Tide Tables for Home and Foreign waters. 4, Coastal Courses and Distances of the British Isles. 5, Lights, Buoys and Beacons of the United Kingdom. 6, Distances from ports in the United Kingdom and the United States of America to many other ports throughout the world, also Navigable Distances between ports as experienced by navigating officers. 7, Miscellaneous information and technical articles on nautical subjects.

The Lights, Beacons and Buoys, British Section, have been revised as far as Notices issued by the Admiralty. Continental Lights, Beacons and Buoys from the River Elbe to Brest are now included. The Tidal Information has been supplied by the Liverpool Observatory and Tidal Institute, permission to use the tidal figures having been given by the Controller of H.M. Stationery Office and the Hydrographer of the Navy.

Courses and distances around the British Isles have been thoroughly overhauled and tabulated. All courses are given Correct Magnetic for 1948 and also True, in three-figure method. Intermediate distances are given to the nearest tenth of a mile.

Coast Protection

A Survey of Beach Stability

By R. R. MINIKIN.

THE ravages of winter storms on the coasts of the British Isles cause annually a considerable amount of repair work to sea-walls, groynes, and beaches. Whereas one may consider these occurrences as inevitable it can be contended, with some justification, that an appreciable amount of the damage could have been avoided. Some substantially built sea defences succumb to quite moderate storms and others, of less sturdy construction, withstand successfully gales of unusual severity. There are reasons for this, which though not obvious, are linked up with established facts of beach maintenance. Some of the mistakes of the past, due to the lack of co-operation of neighbouring authorities responsible for coast protection, are being gradually eliminated. It was not unusual for one authority to be forced to carry out defence works on its own front, to protect it from the damaging consequences of a neighbouring authority's works.

There is now a greater co-operation between neighbouring Boards and Owners, at the same time there are not always equal financial resources. Valuable property may abut directly on land of little or no value where expense of protection would be entirely unwarranted. After all, the economics of any works must be considered; ground values, availability of constructional materials, and labour supply in the district. Notwithstanding the closer co-operation of neighbouring authorities there is, as yet, no evidence of any overall organised plan of defence works on extensive stretches of coast. A not inconsiderable number of works suffering recurring damage are consistently repaired to the original form, or made even more unstable by adding weight in the wrong places. There is a tendency to consider all failures as accidental and due to forces beyond control, instead of searching out and deflecting the causal forces.

What are the established working facts? They may be summarised as follows:—

Causal Forces

(1) Direct wave action.
(2) Temporary drift, due to wave action from the wind quarter, of a few days duration.
(3) Prevailing littoral drift due to wave action as a resultant of a series of storms over a prolonged period, in other words the main tendency of movement along the coast of the loose material on the sea bed.
(4) Concentrated tidal currents sweeping around a headland or breakwater.
(5) The incidence of the wind upon the stability of the beach.
(6) Natural physical formation of the beach slope.

Form of Protection

(7) Solid walls
(8) Submerged walls
(9) Groynes and palisades.
(10) Thatching and fascines.

(1) Direct Wave Action

Though there are various opinions upon the causes underlying the fluctuations of the quantity of loose material on a beach it is now generally agreed that the beach profile is formed by wave action, and the size of the material of which it is composed. Brigadier Bagnold, just prior to the outbreak of the recent war, carried out, on behalf of the Maritime Engineering Section of the Institution of Civil Engineers, some informative and conclusive model experiments on the subject; and formulated for the first time exact definitions of the observed phenomena. The following definitions assume the still water level as constant, that is, there is no tidal range:—

The Beach. He divided the beach into two zones (Fig. 1), the upper beach B, and the lower beach D. The slope with the horizontal of the former is greater than the latter.

Step. The point of separation C, of the two beaches is named the step. At this point there is a decided hump and change of slope.

Beach Shelf. This is the lower portion of the lower beach and is frequently horizontal. Sometimes it rises slightly against the general slope of the beach before merging with the steeper slope of the sea bed.

Beach Crest. The highest point S to which the beach is built up under wave action. It coincides with the upper limit of the wave surge.

Surge Height. The highest point S to which the surge projected by the waves rises on the beach referred to the lowest exposed beach line. This point coincides with the beach crest characteristic of the parent wave.

Beach Angle. This is the angle θ with the horizontal formed by a straight line drawn tangential to the beach crest and the step.

Wave Amplitude. This is the height h of the wave impinging on the beach to give it the profile and is taken as the height from the wave crest (shown dotted) to the lowest exposed beach level.

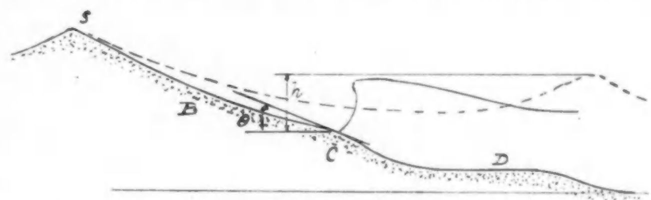


Fig. 1.—Beach profile.

Wave Characteristics and Effects. As the wave from the deep sea progresses over the shallowing shore contours there is first of all a progressive increase of height, trough to crest, and a shortening of the wave length as the water particles in the orbital wave motion "feel the bottom." The period remains constant. However, when the wave length decreases to about seven times the wave amplitude it becomes unstable, and the water particles at the crest, travelling forward more quickly than the restrained particles nearer the bottom, are projected forward from the crest to form a breaking wave. This troubled water is spread over the surface of the sea to the wave front for a short level interval before reforming into waves of lesser amplitude. This goes on successively until eventually the more steeply sloping beach is encountered and there follows a large amount of rapid disturbance. The characteristics of wave formation and stability now lose their mathematical relationship; the wave has now entered the final spending, or dissipation, area of its orbital energy.

The end of deep sea waves takes place close to the shore line in a turmoil of broken water formed by a succession of breaking waves of diminishing length and amplitude until ultimately, the forward wave heaps up to an almost vertical face and projects itself from the crest over the beach in a thinning flush (Fig.2). The tip of this surge reaches to the highest point S (Fig. 1).

Where the foreshore slopes steeply to the sea bed there is a difference of behaviour of the wave dissipation. The velocity of the wave in the deeper water may be maintained to close inshore as there is no time for the natural adjustment to the depth of water, and the gradual spending of the wave, common to flatter foreshores. As a consequence the energy of the wave is spent with greater violence on the beach. This may be observed in particular, on shingle foreshores. It is not unusual in such circumstances to find that the oncoming wave having attained an unstable form does

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not break on the crest but ejects itself from the root up the beach to surge height. The effect on the beach is similar to the crest projection.

Fig. 2.—Waves 10-ft. high breaking on a shingle foreshore.

The progress of the waves from the deep sea to the shore line shows a tendency on meeting the shallows to wheel about the most landward front. In other words, they tend to approach the shore more or less parallel to the beach. The velocity of the waves and the strength of the wind influence the degree of parallelism. In heavy gales and seas coming from a direction oblique to the shore the waves frequently travel too fast for the influence of the shallowing bed to have full effect and the waves plunge obliquely upon the shore. Such seas are productive of the most damaging effects, as the surge sweeps the material of the beach across the slope.

On a beach furnished with groynes it is usual for the breaking wave to occur earlier on one side than on the other of a groyne (Fig. 3). This arises from a series of causes affecting the beach profile arising in the main from the faulty design of the groynes.

(2) Temporary Drift

The passage of a wave over a shallow sea bed causes a degree of commotion in the grains of the surface layers of the bed. They alternate between a moderately close to a loosely packed state under the wave pulsations. Thus, when they are loose they are in a favourable state to be easily influenced by local or temporary currents, be they ever so slight. This is productive of a gradual creep along the bed. The direction of this movement follows that of the waves, tidal currents, or currents of purely local influence, maybe caused by a headland, a rock, or a reef deflecting the main current. There is evidence that in the deeper water this movement of mobile material is continuous, forming deeply submerged bars which may eventually become banks until another cycle of change occurs, as in a gale, bringing about complex and more violent action which sweeps the bank away.

A case in point was the large shingle bank at Totland Bay some 15 years ago. This bank had remained more or less stable for about 7 years, but disappeared almost completely in a few days storm. This continuous building up and destruction cycle of the mobile material has its effect on the neighbouring coast, visible in the amount of erosion, scour, or accretion. The movement is not constant in direction and is, at any one time, dependent on the temporary conditions. There may be depletion of a beach due to the lack of a natural supply of make-up material on the sea bed in the direction from which the storm comes. On the other hand, if there is material on the sea bed close to the coast the likelihood is it will be washed up on the beach. There is an old fisherman's saying "The beach travels through the wind," which has a good deal of truth in it. An offshore wind tends to promote accretion whereas an on-shore gale favours scour of the beach material; there are exceptions however.

An extreme but instructive example of the sea processes of the building up and denuding of a foreshore is given by Sir James Wolfe Barry. He was called in as a consultant by a Government enquiry into the erosion of a beach on the Devon Coast, which had resulted in the partial destruction of a small fishing village. This village was originally, at its nearest point, 80-ft. landwards of the beach line. Sir James reported that a firm of contractors had for some months dredged a large quantity of shingle from the sea below the low water line. As a consequence the hole made in the sea bed set into operation the natural forces to remake the floor and incidentally increased the violence of storm waves on the coast. The deep water over the dredged portion allowed the closer approach to the original crest line of the attacking storm waves. In this example cause and effect can be clearly traced. The regimen of natural formation is a system of local forces in equilibrium which once it is disturbed immediately seeks to right itself at the expense of the nearest vulnerable source of supply. This brings us to the established fact that if erosion takes place at one part of a coast line during a gale, the denuded material goes to swell the supply on the sea bed which provides the accretion in another part of the coast in the direction of the temporary drift. It will be gathered from this, that a gale from any direction may build up, or denude, a beach, depending upon whether there is, or is not, a supply of material on the sea bed in the neighbourhood.

(3) Littoral Drift

On all coasts there is some sector of the compass more productive of gales of long duration than the others, usually, but not always, this is from the prevailing wind direction. It is therefore reasonable to assume, it is also confirmed by observation, that the littoral drift, which can be defined as the resultant travel of the mobile material on the sea bed over the four seasons, takes place in the prevailing wind direction. This movement is not continuous but is accumulative.

Under gales from some sectors there may be a retrogression of the particles for a time, but eventually the predominance of the influencing weather from the prevailing quarter causes a definite forward movement in a settled direction along the coast. In spite of this there may be places on the coast, which have a general littoral drift in one direction, but a local drift in the reverse direction. This phenomenon is due to local configuration of the coast line and sea bed. A case in point is at Chichester where the main littoral drift along the coast is up-channel, that is West to East, but at Chichester Harbour it is East to West. A similar phenomenon takes place on the Coast of Wicklow where the littoral drift is South to North, but owing to the prominence of Bray Head, the local drift in the bay north of the headland is North to South.

Nevertheless, the forward and positive travel of the mobile material over the sea bed supplies the make-up for the beaches of the coast. The progress may be interrupted temporarily by violent

Fig. 3.—The effect of a zig-zag saw-toothed contour of a groyned beach on the break of approaching wave.

storms throwing a quantity of material up into banks on, or over the beaches. This action robs the coast ahead of make-up and thus ensues the phenomenon of erosion or depletion. For example at Blakeney, on the Coast of Norfolk, long stretches of

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shingle banks accumulate to about 6-ft. above H.W.O.S.T. and during severe storms a good deal of this shingle is thrown further inland to be lost to the natural regimen of littoral drift make-up further along the coast. A common occurrence on the South Coast and the Coast of Normandy, is the heaping up of banks of shingle during moderate weather to be later almost completely demolished under stormy westerly gales.

The process of beach building goes on incessantly even during calm weather. In a moderate S.E. wind which had been steady for days and a swell of not more than 3-ft. out at sea, producing wavelets of not more than 18-in. high on the foreshore, the author made a series of tests on the shingle beach at Sandgate, Folkestone.

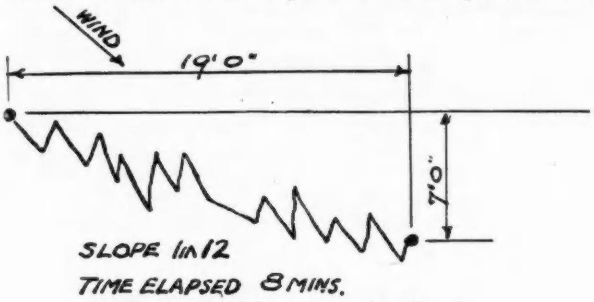

Fig. 4.—Particle movement on beach.

For ease of observation burnt lumps of clay and nuts of coal but slightly heavier than water (specific gravity about 1.3) were used. The travel of these materials among the rounded shingle was easily traceable. By reason of the low density the movement of these pieces was considerably greater than the surrounding beach shingle but the general movement was similar.

These pieces were dropped into 12-in. to 18-in. water depth at about 10 to 15-ft. from the crest line at the time. A typical trace of the path of the lumps is shown in Fig. 4. The movement of each piece was intermittent and jerky, not at all continuous. At times from a position of rest 5 to 8 wave strokes passed before movement again took place. This was not due to any variation of the wave stroke but to the effective cross-sectional area of the piece opposed to the surge flow and the accompanying disposition of the surrounding shingle. It was observed that the individual movement of the shingle was dependent on the shape, flat pieces having a peculiar tumbling motion. During the back wash, the water draining through the voids sometimes dragged the flat pieces on their edges leaving them caught in the other rounded shingle, with the broad flat sides projecting (Fig. 5). The succeeding wave stroke, advancing over the beach, on reaching these pieces carried them forward, and deposited them on their flat faces higher up the beach, where they would remain at comparative rest under a number of wave strokes. They did not always move forward, in fact they followed the pendular motion common to the whole of the beach material, but the forward travel was nearly always greater than the backward.

On the other hand the more rounded pieces were swept forward and then dragged backward, up and down the beach slope with but a slight forward gain for the greater number of movements unless trapped by more irregularly-shaped flatter shingle, whose movements were more intermittent. There appeared to be a selection of the pieces moved, as comparatively few of the pieces in the surface layer moved at any one time. Sometimes a rounded pebble would be pushed forward a few inches then be dragged and rolled back a few feet. Extended observation of loose shingle movement led to the conclusion there is forward (up the beach) movement when the surge flow impinges on the largest projected exposed area of the pebbles when they are in an unbalanced free position occasioned by the turbulence of the broken water. There are also strong grounds to conclude that where there is a predominance of irregularly shaped flat pieces in a beach, it is less subject to depletion than one made up of more spherically shaped particles.

Now if the amount of energy of the surge flow on the return to the sea were equal to the energy it possessed when projected forward over the beach then the material which it pushes up the slope would be dragged down again and the original beach profile would be static. It is obvious this is not the case. On a shingle beach a high percentage of the water fanning out from the surge percolates rapidly through the interstices of the material on its return flow. There is also the tendency for the remainder to be partially dammed up by the rising height of the incoming wave stroke. Thus the energy which the water originally possessed in its mass and velocity is dissipated.

Whether or not a beach is being depleted, or built up, the processes of material movement go on incessantly, and under wave action the tendency is mainly to favour accretion on a sloping foreshore. This fact, although well known, is seldom appreciated at its full importance, as will be seen later. The recent physiographical history of the Coral Island beaches of the Pacific are the ideal examples of this phenomenon. It follows that where a beach is being denuded of its material it is necessary to seek out other likely causes that would favour the drawing down of the material.

(4) Concentrated Tidal Currents

The question of the degree of influence of the tidal current upon the littoral drift has always been productive of argument, some claiming the influence to be great and others inappreciable. However, most are agreed that where there is a high tidal range in estuarial waters there is considerable movement of mobile material, particularly over muddy flats. This is sometimes apparent to the eye, as for example, the muddy waters of the Severn Estuary.

Now the ebb and flow of tides are regular and continuous, but the accretion or depletion of material on a beach is occasional and fortuitious, so it can be reasonably assumed that tidal flow in itself will not influence the regimen of a beach to any great extent, even though the flood has a greater energy than the ebb. The reservations on this point will appear later.

If there is a strong tidal flow along a beach whilst the particles of the beach material are held in suspension by the wave action then there will be undoubtedly a modification of the beach regimen. It is a matter of observation that near the shore landwards of the plunge line of the incoming breakers there is a large amount of material in suspension. The colour of the waters shows this. The translatory effect of the tidal current which is independent of the orbital motion of the waves will therefore carry this material along in the direction of the tidal flow so long as the current is strong enough to keep it in suspension.

On the coasts of the Low Countries the Dutch Rijkswaterstaat, under the directorship of Dr. van Veen, has carried out an extensive research of this, and allied phenomena. It has definitely established that the sand bars and banks in shallow water up to

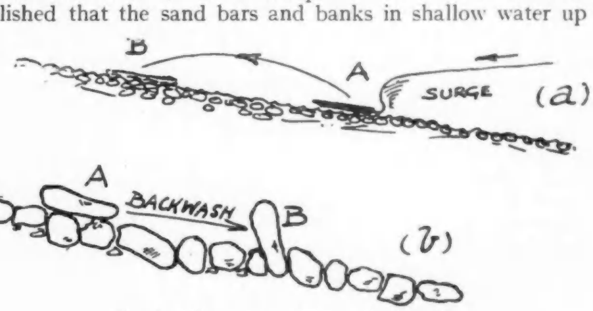

Fig. 5.—Movement of beach shingle.

30-ft. in depth are influenced in their travel along a coast by the ebb and flow of the tides. The flood tide sweeping up the Channel to the north-east hugs the coast from Gris Nez to Gravelines and entering the wide North Sea funnel favours the channels nearer the shore through the sand banks of the Belgian coast. On the ebb the more powerful tidal stream favours the channels to the seaward of the sand banks.

By means of the echo-sounding apparatus the surface contours of the sand banks have been recorded and plotted, and it is found that the general shape of the travelling banks is as shown in Fig. 6. The shape of the cross-sections of these banks shows their direction of movement, the steeper slope being in the direction in which

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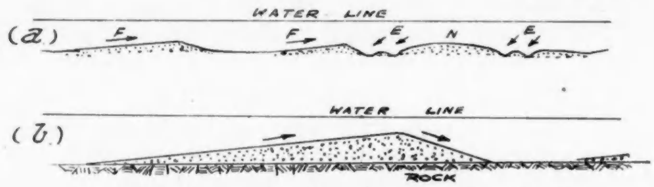

Fig. 6.—(a) Cross-section through series of sand banks. (b) Typical formation of sand bank.

most movement is taking place and consequently the direction of the stronger local tidal current. Those banks which are evenly shaped to the crest or highest point are neutral or have equal movements with the ebb and flow. The plots of the echo-sounding apparatus show that there is a rythmic sequence of sand bars, or sand-waves, of great extent on the sea bed. The constantly changing creep of the crests shows there is a continual drift of enormous quantities of sand along the coast below low water.

In connection with any works which bring about a change in the tidal flow it is interesting to note that Dr. van Veen points out that the sand content (mechanical suspension) of a current varies as the third or fourth power of the velocity; in other words, assuming that unit quantity of sand is transported at velocity v, then at a velocity of 2v the capacity of transportation is increased 8 to 16 times the original quantity. This ratio of sand movement and current velocity naturally applies in reverse, that is, a reduction of the original velocity reduces transporting capacity and causes deposit or silting. Another interesting point which has been confirmed by measurement is the effect of salinity of the tidal stream in estuarial waters. The high density of the flood tide causes it to creep along the sea bed under the less dense waters of the estuary carrying along with it a quantity of silt. On the ebb the lighter upper layers of water flow out first, whereas the denser saline water, on the bottom, remains till the end of the cycle, flowing out with a weak current, thus promoting the deposit of silt. The next flood tide pushes the remaining saline matter still further into the estuary and as Dr. van Veen cautiously remarks, " where a drop of salt water may go, so may a grain of silt."

With the aid of an ingenious simple device the Dutch Tidal Research Engineers established that the smaller sand ripples on the surface of the submerged banks showed by their form the direction of particle movement on the sea bed. The apparatus consisted of two thin metal plates 6-ft. long and 10-in. deep braced vertically at 12-in. apart. They were suspended eccentrically and weights added to balance the edges horizontally. These were given a coat of fresh paint and lowered gently to the sea bed. The eccentric suspension ensured that they would be caused to swing parallel to the current, or perpendicular to the ripples. The sand grains of the bottom stick to the fresh paint and the shape of the ripples show the direction of movement.

As we have already had occasion to remark this gradual and continuous movement of mobile material under the influence of the tidal current does not account for, at times, the rapid depletion or accretion, of the beaches. The most powerful force at work, in this respect, is the energy of the waves. The elliptical orbits of the water particles of a surface wave, near the sea bed in shallow water, become straight line oscillatory horizontal movements. On the passage of a wave over any point on the sea bed the lower layers of water particles shoot forward quickly, and as the wave passes over the point they move backward a lesser amount. Thus in a succession of wave strokes, there is a definite forward movement of the more mobile particles influenced by this action. In the shallower waters towards the beach and up the beach slope the forward movement is intensified (Fig. 7). At the plunge line the

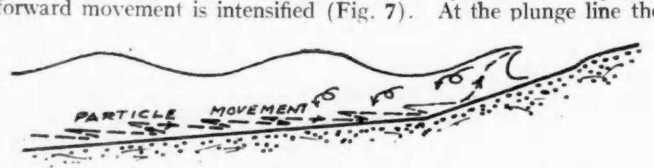

Fig. 7.—Particle movement on sea bed towards a beach.

breaking waves intensify still further the agitation of the water and the suspended particles. In the final turbulence the latter are propelled up the slope of the crest. In spite of the back flow of the surge passing out in the upper layers of the water, the particles in suspension are not carried with it. Once having arrived at the zone of maximum turbulence they do not leave it except to be deposited on the beach. They may, of course, under the purely local influence of tidal or eddy currents be transported along the beach, but under normal conditions the zone of their movements is confined about and above the plunge line. The dotted zig-zag line shows the probable path of a free mechanically suspended particle just skimming the beach surface and eventually jumping up into breaking crest to be projected with the surge, when it may or may not be deposited on the upper beach.

(5). Beach Stability

A naturally 100 per cent. stable beach is rare, except for some comparatively short interval of time. However, granted favourable conditions, it is possible to examine what constitutes beach stability. The general overall definition would be a beach on which the depletion is balanced by an equal amount of accretion uniformly throughout the length. Conditions are seldom, if ever,

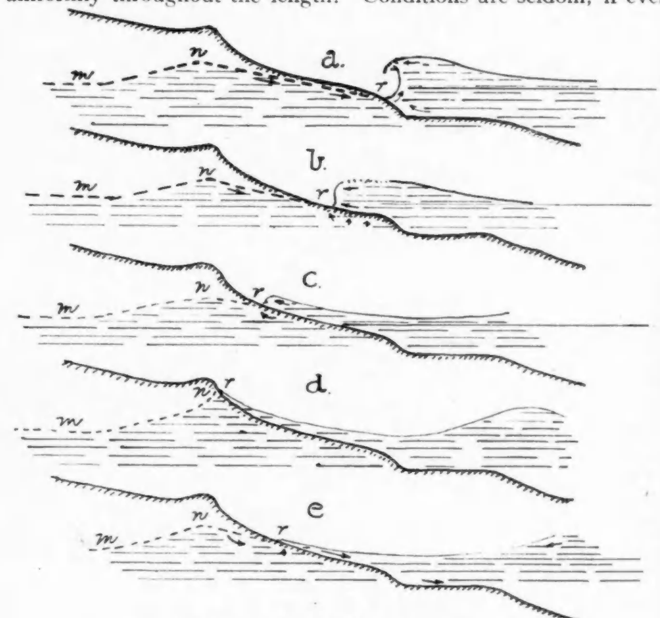

Fig. 8.—Beach building phenomena.

static. There is an incessant change and shuffling of the mobile material even in the calmest of weather.

Assume that the water level is constant; that there is no change of calm weather conditions; and the waves beating on the shore are of slight amplitude which remains unchanged during the time of observation. At the moment when the oncoming wave is just about to break (see Fig. 8 (a)), the wave front stands almost vertically about the position of the step. The return flow of the preceding spent surge has disappeared from the beach surface. The percolating downflow through the shingle has left a water table just under the beach surface indicated by the dash line, m, n, r. The small arrows show the tendencies of direction of the water movement at that instant. It will be noted that the water level within the upper beach is higher than the still water level. After the break the conditions are changed to (b) and in rapid succession to (c) and (d) as the surge travels up the beach to its highest point.

The successive changes in the respective levels of the water table, m, n, r, as the surge advances over the surface and percolates through the shingle (an upward resultant rather than downward) till the crest is reached and the kinetic energy exhausted, causes considerable looseness of the surface layers of the shingle. This looseness favours the transportation of the particles up the slope

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of the crest, particularly those suitably disposed at the point r, the meeting of the water table and the advancing surge. The point r is not necessarily co-incident with the forward tip of the surge. There might be a slight lag dependent upon the degree of porosity of the beach material, in other words, the size and shape of the shingle. When the surge is thinned out to the crest, Fig. 8 (d), there is immediately a rapid return flow of which a considerable portion (e) is drawn down through the shingle. Some of the material is dislodged and dragged down the crest slope and some trundled down the step.

It will be appreciated that the surface material in the forward stroke is subjected to the impress of the kinetic energy of the moving water and a hydrostatic uplift through the beach of diminishing amount as the slope is traversed. This loosening of the material facilitates the forward movement as shown by the small arrows. The larger the projected surface area of the material exposed to the surge the more easily it is pushed to the crest. On the return flow the water careers in a rapidly thinning stream to form the under-tow over the beach. It also percolates downward through the beach. Since the energy of this stream is only due to the height of fall from the rest position and has to contend with higher friction values due to the downward percolation holding the material to the beach it will be appreciated that more material will be moved towards the crest than back to the step unless the beach is mature. In this latter event any material urged forward with the surge will be compensated for by that material drawn down again in the return flow. This in effect means that the limits of beach building have been reached in fashioning the slopes of the beach profile to maximum dimensions under the given conditions.

If the original profile of the beach had been a straight line when the wave action was first commenced, then successions of the above cycle will eventually bring about a profile as shown in Fig. 1, which on maturity, that is physical stability, will remain more or less constant if the height of wave and water depth remain unaltered. An increase in the height of wave however will pick up fresh material at the step and increase the height of the crest. A reduction or increase of water depth will vary the location of the step either lower down or higher up the foreshore respectively. Thus with a falling tide and the same wave height the general slope of the foreshore will be uniform, excepting at the high tide crest.

If along some stable beach an impermeable bulkhead or wall is inserted (A, B, Fig. 10), just in front of the crest and extending into the beach below still water level the dissipation of the surge through the shingle will be considerably restricted. The water table in the beach is reduced to n, r only and the surge tip instead

Fig. 9.—A wave of 9-ft. amplitude breaking at the step of beach.

of climbing the steep slope of the crest material where it would have penetrated the voids of the shingle is deflected upwards on the flat surface A, B. The height of its rise up the wall is about 20 per cent. greater than it would have attained on the porous beach crest. When the forward energy is exhausted the water contacting the wall will collapse and in its fall will scour out the material at the wall as it joins the return flow over the beach. As the water table is close to the beach surface only a small amount of the water passes through the shingle. The consequence is a trench is scoured at the wall; the beach is flattened, and the material is drawn down to the step. The point to note is that the beach building cycle is weakened, if not destroyed, by the restriction of the capacity of the beach to absorb, in its voids, a considerable volume of the surge water. It will be appreciated that the return flow over the surface of the shingle will have a greater destructive effect on the loose particles than if it had been subjected to the damping of its velocity by friction through the shingle voids.

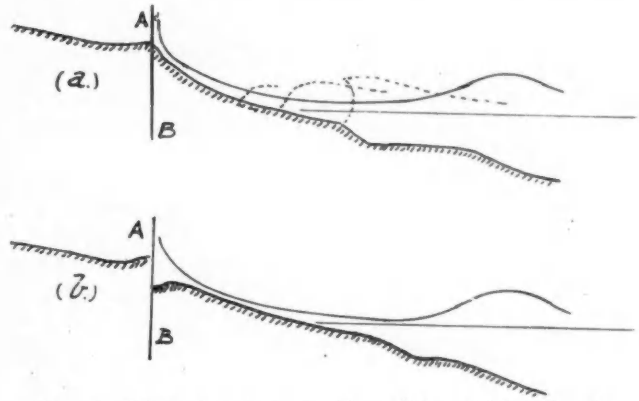


Fig. 10.—Effect of an impermeable wall inserted in a beach.

Here then is the beginning of instability. Had the wall A, B been placed to the landward of the crest the beach would have been stable.

The tendency to scour at the wall under the above conditions may not be uniform over the whole length at one and the same time. On a stretch of beach there are sure to be variations in time when the stroke reaches the wall and there will also be variations in the packing of shingle, although carried out by the same agency. These slight variations, however, give rise to a peculiar selective movement of the shingle; at the loosely packed sections receiving an early stroke the material is scoured away from the wall, but on adjacent sections of slightly higher, or more closely packed shingle, there is a slight building up. On a succession of wave strokes a temporary maturity is reached, but the average level of the beach at the wall is reduced. There may even develop a see-saw or pendular movement of scour and accretion, but it will only last for a short time, or while the conditions remain gentle and constant.

Violent wave strokes, or short choppy seas, will always scour out the shingle at the wall and flatten the beach.

(To be continued)

OBITUARY

Death of Polish Harbour Engineer

We regret to announce the death of **Tadeusz Apolinary Wenda,** well known throughout Poland as the man responsible for the initial planning and construction of the Port of Gdynia. The funeral took place in Warsaw last month, and was attended by the Deputy Minister of Shipping and other officials of that Ministry.

Mr. Wenda was born in Warsaw in 1863, and after receiving his Engineering degree at the Peterbourg Institute, he was engaged for ten years on railway building and subsequently on port construction in Russia. At the conclusion of the 1914-18 war, he returned to Poland, where his wide engineering experience proved of great value to the development of that country. He drew up plans for the expansion of the Port of Gdansk, and until 1937, personally supervised the construction of the Port of Gdynia. Upon retirement, he continued to take a keen interest in port engineering works and his advice on technical problems was always readily available.

The Port of Liverpool

New Dredger Undergoes Trials

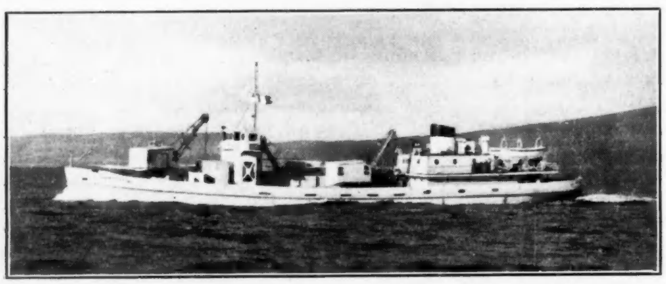
On 11th to 14th October inclusive, "Mersey No. 26," the first of two diesel electric twin screw triple grab hopper dredgers, recently launched by Messrs. Ferguson Bros. (Port Glasgow), Ltd., carried out highly successful trials on the Firth of Clyde. The vessel has been designed and constructed for the Mersey Docks and Harbour Board, which was represented at the trials by Mr. L. Leighton, their engineer-in-chief.

Description of Vessel

The vessel is of more than ordinary interest in view of the constant current system employed for propulsion and dredging, and her general layout and design is on ultra-modern lines. She is classed Lloyd's 100 A.1 and has the following leading particulars:

Length overall	237 feet.
Breadth moulded	40 feet 6 inches.
Draft loaded	14 feet 3 inches.
Hopper capacity	1,350 tons.

The hull has a rounded plate stem and cruiser stern, and the bridge, situated forward, is higher than normal and provides a wide view for navigating and for control of the dredging and unloading operations. Propulsion can be controlled direct from the bridge or by telegraph, and bridge indicators show revolutions and direction of rotation of propellers. A loud hailer is also provided.



The hopper is spanned by a girder carrying the door opening mechanism and a low and high gangway to allow alternative access forward or to bridge, clear of the operational arc of the dredging cranes. On the main deck are also the electrically driven anchor windlass, forward, and windlass, aft. The steering gear, with its electrical equipment, is placed below the main deck directly over the rudder.

Complete accommodation for officers and crew is on and above the main deck. This is contained within a two-level deck house situated above the engine room. The centre of this house is mainly given over to engine room ventilation fans and engine exhaust silencers. On the wings on main deck are two-berth cabins for the crew and similarly, on the upper deck, are the officers' cabins and mess room. Officers' and crew's cabins are provided with hot and cold water. Bath rooms are also fitted for all the personnel. The gallery and pantry are at the forward end of this house extending across ship, on main deck level, with officers' mess room directly above. A service lift communicates between the galley, which is oil fired, and officers' mess. All cabins have artificial controlled ventilation and steam heating with thermostatic control. Cabins are roomy and comfortably fitted.

Independently constructed diesel fuel tanks are in each buoyancy space alongside the hopper. They are easily accessible for inspection and repair, and all electrical leads, running forward, are also easily accessible as they run through these spaces.

Ample deck washing services are arranged for forceful and speedy washing down after dredging.

There is a very spacious hold forward under the bow dredging crane, for stowage of chains and heavy wires. All skylights are internally operated and have armour plate glass. A skylight for engine room lighting is situated inside funnel, clear of silencers.

Three Priestman electrical dredging cranes constitute the dredging units. One is placed at the bow with an operating arc extending from one side, round the bow to the opposite side. There is also a crane on each side abaft the bridge. These two cranes are not placed in a common athwartship line, but are staggered to promote better distribution of the material in the hopper. Either aft crane can discharge forward or aft of its own centre. All cranes can operate to a depth of 70 feet and have a bucket capacity of 70 cubic feet. The immediate discharge of each bucket is on to a grid over the hopper. This grid traps heavy stones liable to damage the hopper doors.

Efficient retention of the dredgings within the hopper is assured by eight all steel construction hopper doors with heavy hinges and closing, all round, on rubber. The hopper door opening and closing mechanism is all of Messrs. Ferguson Brothers' design and it has been carefully planned for smooth and shockless action. A large hydraulic ram is horizontally placed at the forward end of the hopper girder, having the ram at the fore end. This ram moves forward to close the doors. From the ram crosshead, extending aft the full length of the hopper, are heavy section double draw bars running on guided rollers. At each of four positions on the draw bar is attached a chain which runs over the same roller as it passes down to each pair of doors. These single chains break into two chains from the balancing beam. This balancing beam assures equal strain on each door chain. Provision is also made for chain length adjustment. Heavy cutters are fitted for final hardening up of each pair of doors.

The hydraulic ram is charged with special oil pumped from an electric three-throw pump, situated in a forward compartment, and is actuated by push button control at ram control position. The ram is so designed that after pressure has been applied to withdraw the cotters, the whole load in the hopper can be sustained against the medium of the oil in the ram and can be smoothly discharged.

The engine room contains a wide range of duplicated auxiliaries, in addition to the Davey, Paxman main engines and Metropolitan-Vickers generators and motors. A heating boiler burning oil fuel is also installed. The engine room control platform is above floor level and from this platform extends gratings right round the engine room, permitting easy reading of daily service tank gauges and control of high placed valves without the necessity of ladders from main floor level. The piping layout, stern gear and installing of all the machinery has been carried out by Messrs. Ferguson Brothers, and the propeller shafts are protected from the ingress of sand by their "Newark" patent Sterntube oil glands, over 4,000 of which have been supplied to various vessels.

Extensive Trials

The official trials included dredging, off Helensburgh, with all three cranes operating simultaneously and discharging 1,350 tons of heavy silt into the vessel's hopper. This output was easily obtained well within the expected time and the cranes operated perfectly. This full load was retained in the hopper and the vessel proceeded to the Skelmorlie Mile, where full power and speed trials were gone through without a hitch. The results of these runs more than justified the highest anticipations, and these were followed by steering and manœuvring tests, which clearly demonstrated the extreme flexibility and responsiveness of the electrical machinery.

Additional runs were made with only two of the three main generators in use. These runs proved the vessel to be amply powered. During the various tests, complete propulsion control was switched over directly to the bridge controls. The change-over and the actual controlling of propellers from the bridge was carried through with remarkable ease. The vessel subsequently proceeded to the depositing ground and the load of dredgings was discharged quickly and with marked freedom from shock to any part of the discharging system.

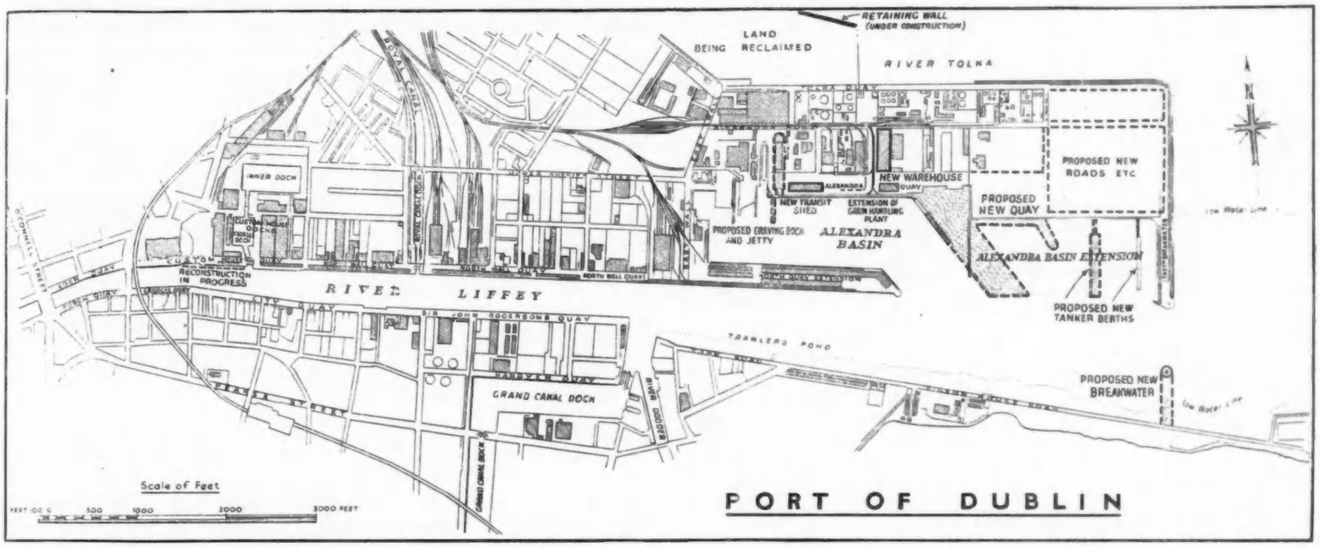
In addition to Mr. Leighton, engineer-in-chief to the Mersey Docks and Harbour Board, the trials were also attended by members of his mechanical, electrical and dredging staff, Lloyd's chief electrical surveyor, members of the staff of Messrs. Davey, Paxman & Co., Ltd., Messrs. Metropolitan-Vickers Electrical Co., Ltd., and Lloyd's local ship and engineering and electrical surveyors.

The Port of Dublin

Improvement Plans in Hand

A detailed account of the reconstruction of the Custom House Quay, Dublin, appeared in the May, 1948, issue of this Journal, and we are indebted to Mr. F. W. Bond, B.A.I., M.Inst.C.E.I., Engineer-in-Chief to the Dublin Port and Docks Board, for further details of the ambitious port extension programme which is at present in hand. The principal works approved are as follows:—

(1) Western extension of Custom House Quay reconstruction work. The second section west of Georges Dock entrance is now in hand.



(2) Deep-water berthage at Alexandra Basin. This will take the form of a spur pier at the eastern end of the basin, providing approximately 1,400 feet of berthage on its west side, 450 feet at its southern end, and 1,000 feet on its eastern side. Only the west side is so far in hand, about 660 feet of quay wall having been completed, while caissons for a further 275 feet have been built and others are building. All this berthage will be constructed of reinforced concrete caissons, founded at about 37-ft. below L.W.O.S.T., with a mass concrete superstructure. Depth of berthage will be 31 to 35 feet at L.W.O.S.T. All berths will be provided with 4-ton electric portal cranes, wagon tramways and transit sheds.

(3) Berthage for oil tankers, Alexandra Basin Extension, consisting of a double sided jetty of similar construction, 590 feet long, providing one berth for large tankers with a depth of 31 feet at L.W.O.S.T. and two berths for coastal tankers with depths of about 20 feet at L.W.O.S.T. This work is now nearing completion.

(4) Berth for coastal oil tankers at Pigeon House Power Station for the Electricity Supply Board, consisting of timber dolphins and access bridge. This berth will have a depth of 23 feet at L.W.O.S.T.

(5) Reclamation for industrial sites at East Wall. This area is being reclaimed by tipping of city refuse—the retaining wall, which is built on small un-reinforced concrete caissons, being constructed by the Port Board.

(6) Crane installations: 26 new electric portal cranes of 4-ton capacity have been ordered from Sir Wm. Arrol & Co. for equipping the new berths and replacing obsolete cranes. The first three cranes are now almost complete and further consignments will arrive shortly. In connection with these, two new rectifier sub-stations are being constructed.

(7) New Transit Sheds: A new double storey transit shed, approximately 380-ft. by 80-ft., is being constructed at Alexandra Quay and additional sheds of the same type will be built as the new berths are completed. In connection with this scheme, the privately-owned grain discharging plant at Alexandra Quay has been altered and extended to cover an increased length of berthage. It is also intended to build additional sheds on the North Quay Extension.

(8) New Warehouse: Foundations for a four-storey reinforced concrete general warehouse at Alexandra Quay has been nearly completed and tenders for the main building have been invited. This warehouse will cover an area of about 470-ft. by 150-ft. and will be equipped with goods lifts and package elevators in all compartments. It will have direct rail connections.

(9) New Graving Dock: Plans for a new graving dock, 630 feet long, are practically complete and tenders will probably be invited in the near future.

(10) Deepening of the Bar: Dredging of the bar to obtain an increased depth in the approach channel is in progress. This work is being done by contract.

A number of minor works are also in hand. Other improvements which will be undertaken as soon as conditions permit are construction of two additional deep berths for oil tankers; development of the reclaimed area north of Alexandra Basin Extension; and construction of a short breakwater on the south side of the river channel opposite to the eastern breakwater.

Other proposed developments include new bridges and new quays on the south side of the River Liffey, but the plans for these are not yet definite.

The principal works in hand and contemplated are indicated on the above plan of the port, and it is expected that Papers dealing with the principal works will be presented to the Institution of Civil Engineers of Ireland when the works are completed or have reached a stage to justify a full description.

Siamese Port Developments.

It has been announced by the Ministry of Communications that the Government of Siam is to invite tenders from private contractors to develop and operate the harbour facilities of the Island of Koh Sichang on the mouth of the Bangkok River, about 20 miles south of the capital. Private interests will also be permitted to improve and operate the Ports of Singora (Songkhla) Puket and Kantang in the rich tin and rubber-producing area of the Siamese section of the Malay Peninsula.

These port development plans are designed to foster the country's foreign trade and the concessions are being offered to private firms, Siamese or foreign, because of the lack of funds and technical knowledge in the country.

Several Siamese and foreign firms have expressed interest in the scheme, although the terms of the concession offered have not yet been officially announced.

The Construction of Cofferdams

An Article for Students and Junior Engineers

By STANLEY C. BAILEY, Assoc.M.Inst.C.E., F.G.S.

General

COFFERDAMS, so named from the Mediæval English word "cofer," meaning a chest or box, are enclosed temporary dams, usually constructed of a single row of steel or timber sheet piling, with walings, braces, and struts on the inside, steel sheet piles are inter-locked or keyed together, but timber piles may be used with or without keys; they are driven from 10 to 20-ft. or more into the sea or river bed, according to the depth of excavation required inside the dam, and may be either square, rectangular or circular in plan, with sometimes the addition of triangular cut waters at the up and down stream ends of square or rectangular ones.

Temporary dams are usually designed by the contractor for the permanent work, but their design and construction must be approved by the engineers responsible for that work. Cofferdams have been formed of a double row of timber or steel sheet piling spaced from 5 to 8-ft. apart, with inside and outside walings, and cross tie bolts, the space between the sheet piles being filled with concrete or puddle clay, free from stones; this is an expensive type, and is seldom used at present in the form of a cofferdam, but in cases where large areas, such as for the construction of new basins or dry docks, are required to be cut off from the sea, to enable the work to be done in the dry, and where clay is readily available, this type of dam is often used. Enclosed dams formed of a single row of steel or timber sheet piles, are chiefly used in connection with the construction of bridge piers in a river, and the building of the under-water portions of ship slipways, and occasionally for surrounding a sunken ship in a harbour, so that it may be repaired for refloating, in which case the dam is strutted against the sides of the ship.

The term "box" or limpet dams is applied to portable three-sided or semi-circular ones, with bases, and formed of steel or timber, that are lowered by a crane against the walls of a wet dock, for alterations and repairs to be carried out in the dry, such as the fixing of new fender timbers.

Materials

There are various forms of steel inter-locking sheet piles used, and the names of the manufacturers of the various types can be obtained from the advertisements in this journal.

The tearing apart strength of the vertical joints varies with the type of pile used, the strongest are those formed of rolled steel joists, or a web with bulged flanges inter-locked by steel joist needles with the flanges turned inwards to grip the bulges, and weighing about 43 lbs. per sq. ft., these have an ultimate strength of 8,370 to 10,900 lbs., or 3.37 to 4.86 tons respectively per vertical inch of pile. The width between the joints varies from 8 to 22-in., according to the type of piling, and no shoes or special points are required for soft ground, but for hard driving the piles are sometimes cut with triangular points. These piles may be obtained in lengths up to 50, 60 or 70-ft., and the walings and struts used are generally of timber. The wood used for timber piles, walings and struts is usually, either Dantzic, Riga, or Memel redwood and yellow pine, but pitch pine, Oregon and Douglas firs and pines are also employed, the softer timbers will swell in water, and thus assist in keeping the joints watertight.

The scantlings of the piles may vary between 9, 12, and 14-in. sq., with lengths up to 70-ft., they should be straight, or free from wanes or bends, and cracks or shakes, and loose knots; the longest timbers are usually obtained from Oregon or Douglas fir. The ultimate breaking strength of some of these woods, based on tests made on large specimens in tension (T) and end compression (C) in the per sq. in., is as follows, viz:—Dantzic, Riga and Memel redwood T = 10,000, C = 6,000.

Yellow pine T = 10,000, C = 8,000; pitch pine T = 12,000, C = 8,000, Oregon and Douglas pine, T = 12,000, C = 6,000, and Norway pine, T = 8,000, C = 6,000.

Pile Fittings and Shoes

Timber sheet piles may be driven either with or without being keyed together, but for watertight work, it is advisable to use keys, these are formed by cutting a 3-in. by $1\frac{1}{2}$-in. deep central groove on one side of the pile, and on the opposite side, so cutting the pile that there is a 3-in. by $1\frac{1}{2}$-in. tongue, to engage the groove on the next pile; or by cutting grooves on the opposite sides of each pile, and driving 10-ft. lengths of 3-in. by 3-in. hard wood needles into the groove. These methods involve considerable saw work on the piles and waste of timber; a simpler method, say, on a 12-in. sq. pile is to spike two strips of wood $4\frac{1}{2}$-in. by $1\frac{1}{2}$-in. on one side of the pile leaving a 3-in. space between them, and on the opposite side to spike a 3-in. by $1\frac{1}{2}$-in. hard wood centre strip to form a tongue, as shown in Fig. 1. Another method used, where whole timbers are not readily available, is to bolt three planks about 12-in. by 3-in. or 4-in. together, the central plank projects $1\frac{1}{2}$-in. beyond the others, so that a tongue is formed on one side, and a groove on the opposite side. The bolts may be $\frac{5}{8}$-in. or $\frac{3}{4}$-in. diameter, and arranged zig-zag at 3-ft. pitch, as illustrated in Fig. 2.

The top of the pile should be adzed down to form a circle 2-in. in diameter less than that of the pile, on which is fitted a $2\frac{1}{2}$-in. by $\frac{3}{4}$-in. or 3-in. by 1-in. mild steel or wrought iron ring. A number of rings will be required as it is not always possible to easily remove one, unless the head of the pile is sawn off just below the ring.

A hole near the head of the pile will be required for a toggle bolt $1\frac{1}{4}$-in. diameter, to hold the pile against the pile frame leaders. When driving the piles in mud or soft soil, the only shoes required, may consist of a $\frac{3}{8}$-in. or $\frac{1}{2}$-in. thick steel plate bent to a V shape and about 9-in. high, spiked to the wedge-shaped point of the pile, as shown in Fig. 3, but for harder ground, special cast iron shoes with chilled points, fastened to the pile by four $2\frac{1}{2}$-in. by $\frac{3}{8}$-in. mild steel straps, and $2\frac{1}{2}$-in. by $\frac{3}{8}$-in. diameter spikes (see Fig. 4) are fitted. The bearing area of the timber on the shoe should be so large as possible, and the straps should be bent to a U shape, and embedded in the mould of the shoe, so that they are firmly fixed in the shoe when it is cast. If attached by rivets, they are very liable to be torn away, especially if the pile strikes a large stone.

Sheet pile shoes have wedge-shaped points, the apices having a slope of from 1 in 6 to 1 in 8, the front of the shoe is vertical, but the back has a slope of about 1 in 3, these slopes cause the piles to close up against one another while being driven. The following are the approximate weights of sheet pile shoes and straps, viz.: 9-in. by 9-in.—42 lbs., 12-in. by 12-in.—66 lbs., 14-in. by 14-in. —93 lbs. Ordinary plug and gauge or king piles have shoes as shown in Fig. 5 with pyramidal points, attached to the pile by four mild steel straps and spikes; in some cases a $\frac{3}{4}$-in. deep sinking is formed in the timber bearing area of the pile shoe, but this is scarcely necessary.

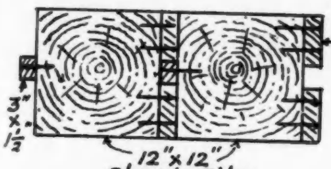
The weights of plug pile shoes are as follows, viz.: 9-in. by 9-in. —32 lbs., 12-in. by 12-in.—40 lbs., and 14-in. by 14-in.,—58 lbs.

The slope given to the point of the pile is usually about 1 in 4, and it is not necessary to tar or paint the shoes and straps.

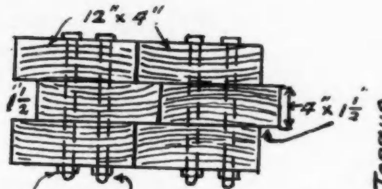
Driving Piles

The piles for cofferdams are usually driven from pile driver frames mounted on a floating pontoon, or decked barge, moored at the site, and fitted with steam, Diesel oil, or electrically-driven winches and engines, the weights of which balance the pile-driving frame and hammer.

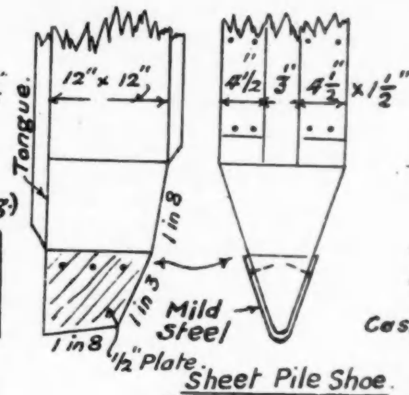
Construction of Cofferdams—continued



Sheet Piles.
Fig. 1.



¾"d. Bolts. 3'-0" apart (zig zag.)
Fig. 2



Sheet Pile Shoe.
Fig. 3.

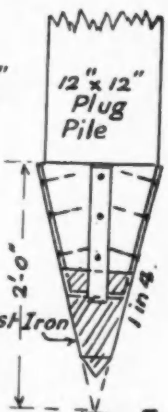


Fig. 5.

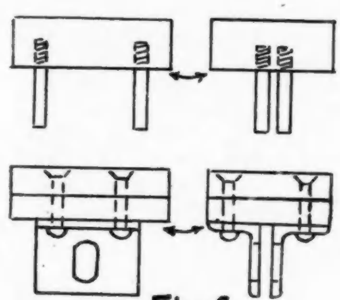
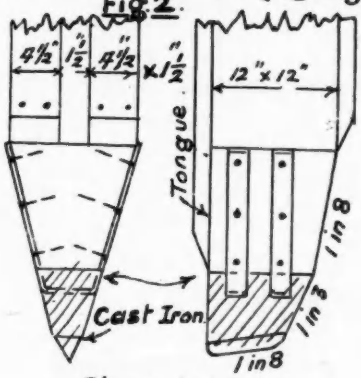


Fig. 6.
Steel Caps For Steel Sheet Piles.



Sheet Pile Shoe.
Fig. 4

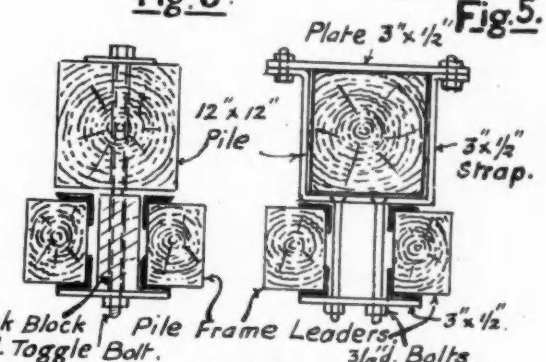
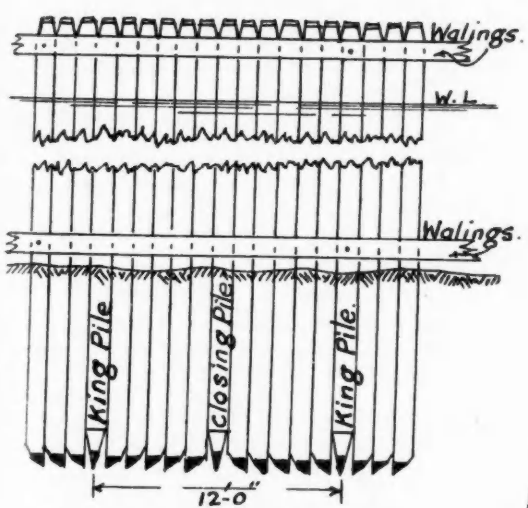


Fig. 7.



Elevation of Sheet Piling.
Fig. 8.

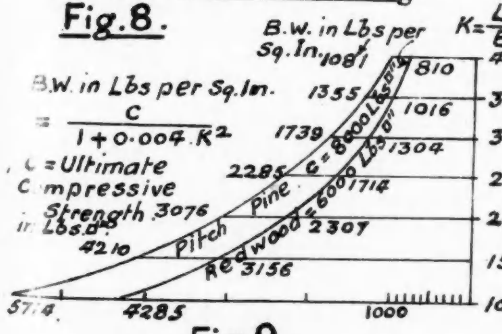


Fig. 9.
Breaking Strength of Struts.

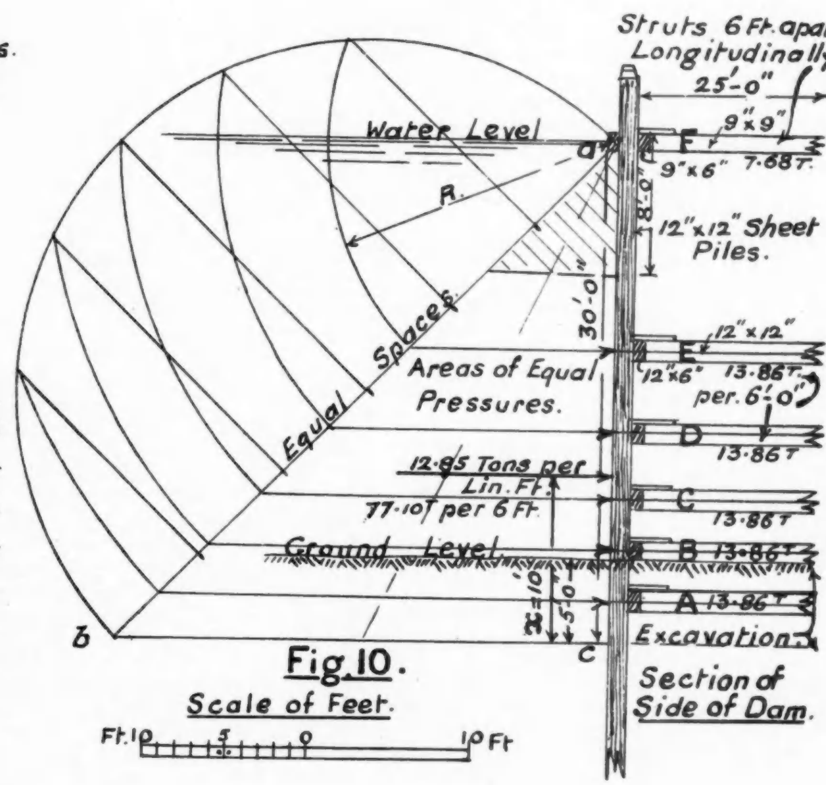


Fig. 10.

Construction of Cofferdams—continued

The hammers or rams used, may either be steam-driven or pneumatic, delivering from 50 to 200 blows or more per minute.

When steel sheet piles are driven into boulder clay, or gravel full of large hard stones, care should be taken that they are not driven too hard, as the points of the piles may strike a boulder and be deflected, causing the pile to bend, and appear to go down easily, when it is actually turning outwards or inwards.

If a boulder is encountered, the set will be suddenly reduced, and should repeated blows fail to pierce it, the pile should be withdrawn, and a long steel rod about 3-in. in diameter with a wedge-shaped or chisel point, driven down to split the boulder.

When steel or timber sheet piles are driven into a considerable thickness of soft soil and mud, there may be some difficulty in keeping the joints of the dam watertight, and the water may also force its way under the dam, causing bad "blows," which may even lift a timber dam; in which case the joints should be caulked with oakum on the inside, as the water is being pumped out, and a bank of clay tipped close to, and around the outside of the dam, this will gradually, effectively seal the leakage. The dumping of sawdust and wood pulp outside the dam has been found effective in closing the pile joints, and where possible the piles should be driven down to a harder stratum.

When the piles are driven into chalk, or limestone and sandstone, it is advisable to first cut a groove in the rock about one foot or more deep with a steel rod having a wedge-shaped point, then drive the piles until they stick in the groove, it may be necessary also to deposit some concrete against the feet of the piles, both on the inside and outside.

When there is difficulty in keeping the piles vertical while driving, an additional toggle bolt should be fixed several feet below the top bolt, the former can be removed when it reaches the base of the pile frame or extended leaders.

On no account should chains round the pile be attached to the pile frame, as is sometimes done, for these arrest the descent of the pile, and will cause damage to the pile-driving frame. When driving timber piles no special caps, except the steel ring, are required, but should the pile heads become shattered or crushed in hard driving by the ram, an oak "dolly" about 2-ft. long, with rings on both ends may be superimposed on the pile head, but this reduces the force of the blow to about half. In the case of steel piles, forged or cast steel anvil blocks are fitted by clips or studs to the pile head to prevent damage, and in Fig. 6 two forms of these anvil blocks are shown.

Fig. 7 illustrates two methods of holding the pile against the pile frame leaders, in one a toggle bolt is used, and in the other, mild steel straps round the pile, to avoid making a toggle hole, as this sometimes leads to splitting of the pile head during hard driving.

Construction of Dams

If the cofferdam is required for a ship slipway sea end, or a bridge pier, the king or gauge plug piles spaced about 10 or 12-ft. apart at the side of the dam, are first driven, and inside and outside top walings are spiked or bolted to these; similar walings at the sea or river bed level are also fixed when timber piles are not keyed together.

The sheeting piles are then driven between the walings on each side of the king piles, and will tend to wedge themselves against them, and to one another; when the centre of a 10-ft. or 12-ft. bay is reached there will probably be a gap left, which must be filled with a specially cut closing plug pile to fit the space, as illustrated in Fig. 8.

When the piling has been completed, the pile heads may be from 3 to 4-ft. or more above the highest tide level, and the pumping out of the water inside the dam may be begun. As the water level falls, the inside walings and struts are fixed, the walings being secured to the king piles by spikes, sometimes short blocks of wood or chocks are spiked under the walings as supports. The water should not be pumped out too much in advance of the fixing of the walings and struts at the different levels, and all leaks at the joints should be caulked as the water level falls.

The horizontal struts between the walings on opposite sides of the dam, should be square timbers, butting close against the walings, if not, hard wood wedges should be driven in between the ends of the struts and the walings; in addition the struts are fixed to the walings by short lengths of capping timbers on top, and about 2-in. or 3-in. thick spiked to them.

In some cases mild steel or iron angle bars 12-in. by 12-in. by $2\frac{1}{2}$-in. by $\frac{1}{2}$-in. are fixed by spikes on each side of the struts to attach them to the walings, but these cannot readily be removed when no longer required.

In the case of circular cofferdams with timber piling, the meeting faces of the piles will require to be cut to radial lines if the dam is less than 20-ft. in diameter.

Steel piles may be obtained with the webs bent to almost any radius to form pit shafts or for use as hollow single piles. The joints of steel piles are sometimes well lubricated with axle grease or thick oil before driving, this assists in keeping the joints watertight, and makes the driving easier.

Vertical timber bracing fixed diagonally between the tiers of struts is not necessary in cases where the ground is clay or compact sand, but in mud or alluvium it may be required if there is unbalanced pressure on the sides of the dam, due to the percolation of water to a greater depth on one side of the dam, than on the other, with a tendency to uplift.

In steel sheet pile dams, when the horizontal distance between the tiers of struts exceeds 8-ft. steel joist walings are sometimes fixed, they are suspended from the top by wire ropes or steel rods, until the struts are fixed against them, the struts are attached to them by steel angle cleats. Short vertical lengths of timber known as "puncheons" support the walings against the sheet piles.

When the interior of the dam has been pumped dry, the excavation for the foundations of the slipway end, or bridge pier, can be proceeded with, the materials being loaded into tipping buckets or skips with hinged bottoms, slung from a floating crane outside the dam, which deposits the materials into a hopper barge alongside; or the soil may be loosened by the workmen and lifted by a clam dredge bucket suspended from the crane.

As the excavation proceeds, additional walings and struts will require to be inserted between the sides of the dam, as the water pressure will then be the heaviest.

When the permanent structure of a pier is sufficiently advanced the struts of the dam are removed where they interfere with the permanent work, and shorter struts are inserted between the sides of the dam and the pier.

Pumps

The pumps may either be of the centrifugal, turbine, or piston and plunger type, driven by Diesel oil, steam or electric power, submersible electrically-driven centrifugal pumps are much used. In some cases the pumps are fixed to a platform over and at one end of the dam, and in others they are mounted on a barge moored alongside, the suction pipes being either flexible hose or steel tubes with a "rose" at the lower end.

Removal of Dams

When the permanent work has been completed, the removal of the dam may be begun, by first taking off the capping pieces to struts and walings, and the angle iron connections, then a few sheet piles are withdrawn to flood the interior, the remaining sheet piles are pulled, and lastly the king and closing piles, the walings and struts will then float to the surface.

If the piles are in soft ground, they may be withdrawn by the pile frame rope attached to a bolt in the toggle hole of the pile head, or by a chain loop over the pile head and under the toggle bolt, or to cleats spiked to the pile, this method is often used for pulling steel sheet piles, which have only about half the frictional area in the ground that timber ones have, but it puts a considerable strain on the pile frame, so that special pile extractor machines are now much used; one of these consists in reversing a steam or pneumatic pile-driving hammer slung from the pile driver frame, the hammer delivers its blows upward against an anvil block attached to side links or steel bars connected at the lower ends to special jaws or clamps with wedges, which grip the pile head.

Another form is a specially made extractor machine very similar to, and working on the same principle as the reversed pile-driving

Construction of Cofferdams—continued

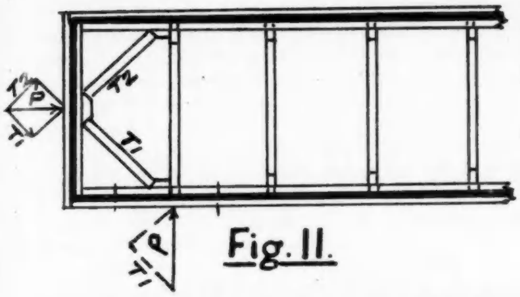


Fig. 11.

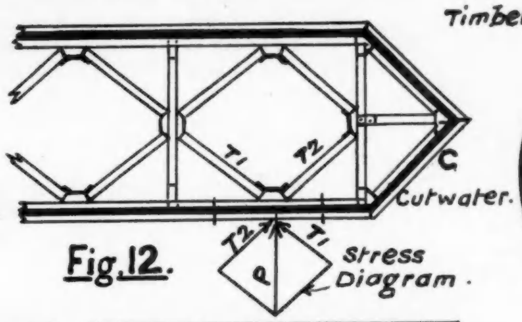


Fig. 12.

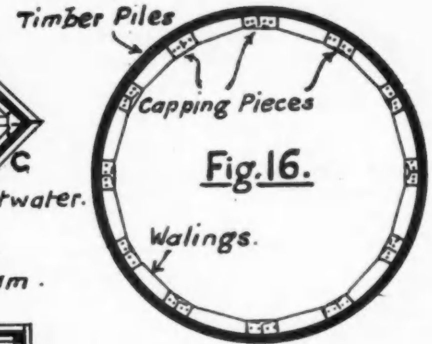


Fig. 16.

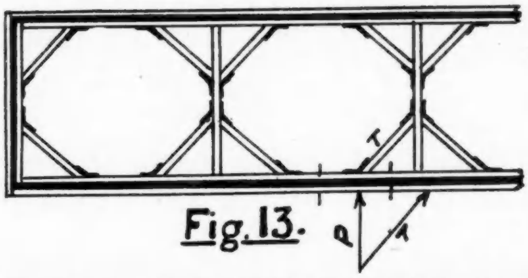


Fig. 13.

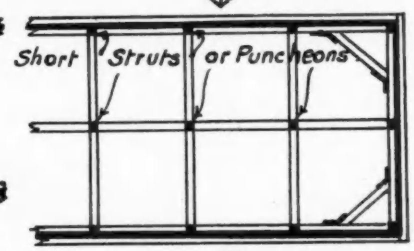


Fig. 14.

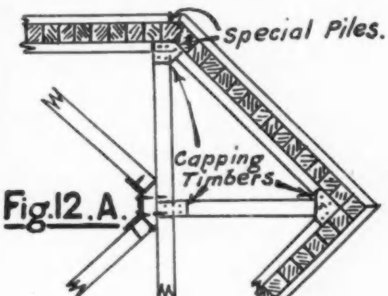


Fig. 12. A.

Detail of Cutwater at "C".

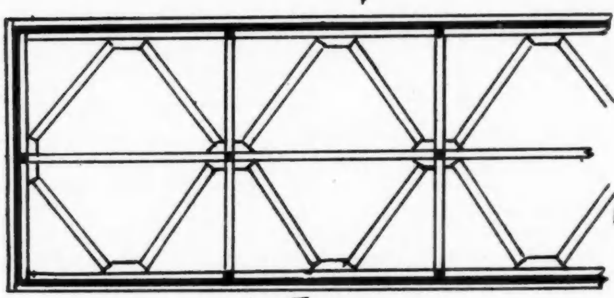


Fig. 15.

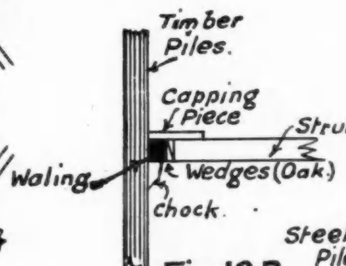


Fig. 12 B.

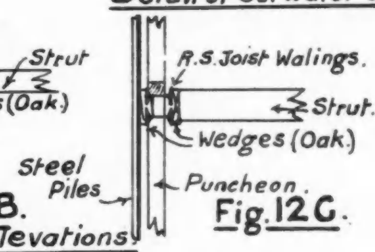


Fig. 12 C.

Elevations.

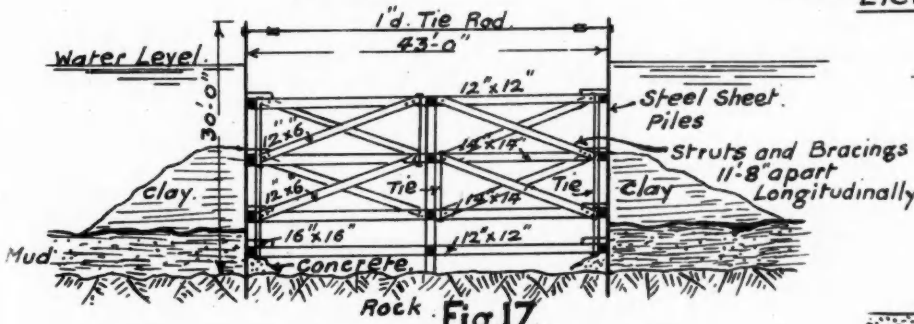


Fig. 17.

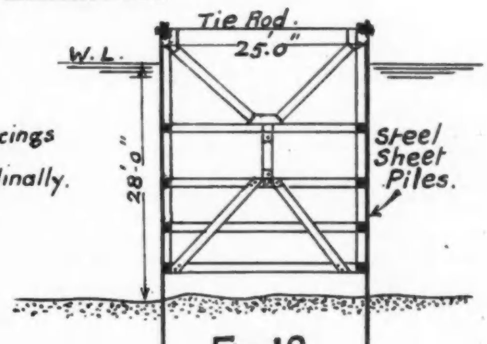


Fig. 18.

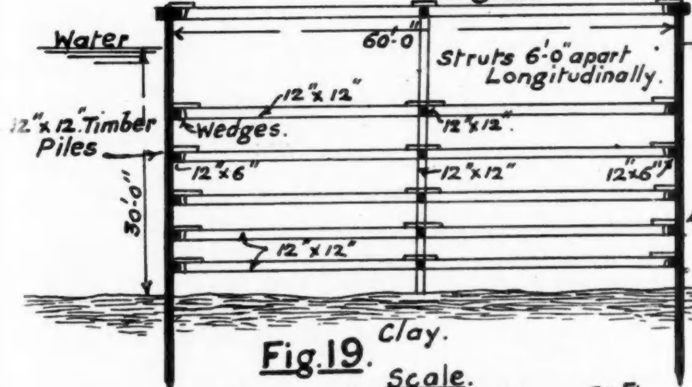


Fig. 19.

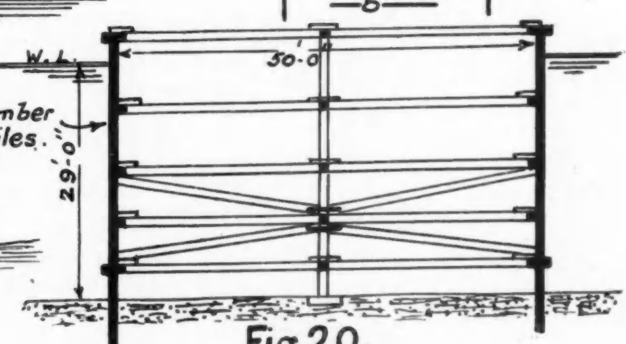


Fig. 20.

Construction of Cofferdams—continued

hammer. These extractors after the first few rapid blows to loosen the pile, will pull them very rapidly. When the timber sheet piles of a cofferdam are removed from stiff ground such as clay, a trench will be left round the permanent structure, as this is very inadvisable, because the weight of the pier will probably cause the ground to spread, so the trench should either be filled with gravel or concrete, or the piles cut off at the sea or river bed level by divers using oxy-hydrogen, oxy-acetylene, or electric cutters, designed for under-water work. Some wharf walls have failed through trench sheet piles having been withdrawn, causing the walls to slide forward and crack. If steel sheet piles are used, they can be safely withdrawn, without much disturbance of the ground, but if the concrete foundation of the pier extends to the sheet piles, then at the level of the top of the concrete, the piles are jointed by plates and bolts, so that the upper portions may be removed without disturbing the lengths in the ground, and so avoid cutting them.

Calculations

In constructional work, timber is an uncertain material as regards strength, so much depends on whether it is a soft or hard wood, wet or dry, unseasoned or seasoned, and free from cracks and loose knots, while hard woods are inclined to be brittle, so when calculating the strength of timber for cofferdams, which is more or less saturated with water, due allowance must be made in the factor of safety used, although in works of a temporary character a lower factor of safety may be taken, than would otherwise be advisable.

When dry timber is employed in building construction a factor of safety of 10 is usually allowed for tension, 5 for end compression and 4 for shear.

Timber is capable of holding from 8 to 15 lbs. and more, of water per cub. ft., the fibres swell when it is wet and reduce the strength, but in dry wood they contract, and the strength is increased by from 10 to 12% in soft woods and 25 to 40% in hard ones. As regard the scantlings of timber, Memel redwood can be obtained in baulks up to 14-in. sq., and Dantzic fir up to 18-in. and more, while Douglas, Oregon and Washington pines are obtainable in long lengths and large scantlings.

The diagram in Fig. 9 shows the ultimate crushing strength of pitch pine and redwood struts in lbs. per sq. in., based on C. Shaler Smith's formula in which the breaking strength of struts in lbs. per sq. in. $= \frac{C}{1 + 0.004K^2}$ where C is the ultimate compressive strength in lbs. per sq. in. of short lengths, and $K = \frac{L}{B}$, in which L = the length and B = the least width, in similar units.

The pressures on dams should be calculated for more than the head of water to the bed of the sea or river, as the water will creep down several feet against the piling below that level, so that the actual pressures to be sustained are uncertain, depending much on the character of the soil; in clay and compact sand there may not be much extra pressure, but in mud, gravel, and loose sand it may be considerable, and extend to the shoes of the piles.

The sketch in Fig. 10 shows one side of a proposed cofferdam 25-ft. wide, formed of timber sheet piling, walings, and struts. The head of water to the bed of the sea is 25-ft., and 5-ft. extra has been allowed for a possible excess pressure, the total head of 30-ft. will give a pressure of $\frac{30' \times 30' \times 1'}{2 \times 35} = 12.85$ tons per lin. ft. sea water being taken at 64 lbs. per cub. ft. or 35 cub. ft. per ton, which acts at the c, g of the equilateral triangle a, b, c, or 10-ft. = x from the base.

Divide the hynotenuse a, b into, say 6 equal parts, and construct the diagram as shown in the sketch, the triangle will then be divided into areas of equal pressures, at which the struts can be placed, say 6-ft. apart longitudinally.

The pressure on the top strut F due to the hatched triangular load = 1.28 tons per lin. ft. $\therefore 12.85 - 1.28 = 11.57$ tons on the 5 lower struts, or 2.31 tons per strut, which by 6-ft. = 13.86 tons, and 7.68 tons on the top strut.

There will be a similar pressure on these struts from the opposite side of the dam, but as action and reaction are equal and opposite, the pressure on the struts will still be 13.86 tons, unless there is unbalanced water pressure, in which case the effective pressure on the strut will be the greater of the two pressures. The pressures on the struts can be reduced by allowing the timber piles to safely carry a proportion of the load, but this cannot be done with steel sheet piles, as their resistance to bending will not be great.

The strength of a 12-in. by 12-in. timber pile acting as a cantilever will be as follows, viz.: the B.M. = wx = 12.85 by 10-ft. = 128.5 ft. tons = 3,454,080 inch lbs.

The moment of intertia $I = \frac{B, D^3}{12} = \frac{12 \times 12^3}{12} = 1{,}728$-in. units, and y = half the depth of the pile = 6-in.

Therefore the stress will be $= \frac{M.Y.}{I} = \frac{3{,}454{,}080 \times 6''}{1728} = 11{,}993$ lbs. per sq. in. which will be the breaking strength of the pile.

Allowing the pile to sustain safely a B.M. of 431,760 in. lbs. or $\frac{1}{8}$ of the maximum strength, the safe load will be $= \frac{M}{x} = \frac{431{,}760}{10' \times 12''} = 3{,}598$ lbs. = 1.6 tons.

As the struts are 6-ft. apart longitudinally $\therefore 1.6 \times 6' = 9.6$ tons is the reduction of pressure on 6 struts in each tier, or 1.6 tons per strut, therefore the net pressure on the top strut $F = 7.68 - 1.6 = 6.08$ tons, and on each of the remaining struts $= 13.86 - 1.6 = 12.26$ tons.

With regard to the strength of the struts 24-ft. long between the walings, assuming a 12" × 12" strut, and using the formula already given, the B.W. in lbs. per sq. in. $= \frac{C}{1 + 0.004.K^2} = \frac{6{,}000}{1 + 0.004 \times 24^2} = 1{,}818$ lbs. per sq. in. and $1{,}818 = 12'' \times 12'' = 261{,}792$ lbs. = 116.8 tons; and as the maximum load is 12.26 tons, there will be a factor of safety of 9.5.

The top strut F can be reduced to 9" × 9", as the load is 6 tons and the B.W. is 1,200 lbs. per sq. in., which by 81 sq. in. = 97,200 lbs. = 43.3 tons, giving a factor of safety of 7.2. The length of struts should not exceed 30 times the least breadth where possible, without central supports both underneath, on top and longitudinally in the length of the dam.

In the case of the walings say 12" × 6" × 6-ft. span, they may be treated as beams with fixed ends, under a distributed load of 12.26 tons; the net B.M. at the centre $= \frac{W.L.}{24} = \frac{12.26 \times 6'}{24} = 3.06$-ft. tons $= 82{,}252.8$ in. lbs. and at the ends the B.M. $= \frac{W.L.}{12}$ 164,505.6 in. lbs.

$$I = \frac{B.D^3}{12} = \frac{12 \times 6^3}{12} = 216 \text{ in. units.}$$

The stress at the centre $\frac{M.Y.}{I} = \frac{82{,}252.8 \times 3''}{216} = 1{,}142.4$ lbs. per sq. in., and the factor of safety $= \frac{10{,}000}{1142.4} = 8.7$.

The stress at the ends will be double this and equal to 2,284.8 lbs. per sq. in., giving a factor of safety of 4.35 which is rather low, but it will be safer against shearing. The shear at each end $\frac{W}{2} = \frac{12.26}{2} = 6.13$ tons, and the ultimate shearing strength of the following woods across the grain in lbs. per sq. in., is for Georgia yellow pine—5,000, Northern (American) yellow pine—4,000 and Californian redwood—1,600.

Construction of Cofferdams—continued

Assuming 1,600 lbs. × 12-in. × 6-in. = 51.4 tons maximum shearing strength, there will be a factor of safety of $\frac{51.4}{6.13} = 8.5$ and a factor of 4 is usually allowed. With regard to circular dams, their strength may be calculated as follows, viz:—Assuming a dam of 30-ft. in diameter under a head of water = 30-ft. the pressure per sq. ft. at the bottom will be $\frac{30' \times 1' \times 1'}{35} = 0.857$ tons, and the circumference $= 30 \times 3.14 = 94.2$-ft. the total pressure on the dam $= 0.857 \times 94.2 = 80.72$ tons. If half the dam is treated as an arch, the pressure will be 40.36 tons, and the horizontal thrust $\text{H.T.} = \frac{\text{W.L.}}{8.\text{v.}} = \frac{40.36 \times 30'}{8 \times 15} = 10$ tons per sq. ft. where W = the total pressure L = the span, and V = the verse sine, or radius in this case.

Assuming timber sheet piles 9-in. by 9-in., the area at the lower foot will be 12-in. by 9-in. = 108 sq. in., and 10 tons = 22,400 lbs., so the pressure = 207.3 lbs. per sq. in.

The ultimate crushing strength of some timbers across the grain in lbs. per sq. in. is for Georgia yellow pine—1,400, for Douglas, Oregon, and Washington fir—1,200, and for Memel redwood, Norway pine, and Californian redwood—800 lbs. thus at 800 lbs. per sq. in., there will be a factor of safety of 3.85, and one of 4 is usually taken. There is not much difference between the crushing strength of hard woods with and across the grain. If the sheet piles are of steel with webs ⅜-in. thick having an area of 12-in. by ⅜-in. = 4.5 sq. in., the pressure will be 2.22 tons per sq. in., this will be a safe one, provided the web is not very wide, and has no curve in it, which will not be necessary in a diameter of 30-ft.

Types of Cofferdams

Figs. 11 to 15 are sketch plans of rectangular cofferdams showing the various methods of strutting employed, the pressures that the diagonal struts will have to sustain are shown by the stress diagram in Figs. 11, 12 and 13, and where there is a strut in alignment with the pressure, and also a diagonal as in Fig. 11, the strut in line with the pressure will carry the whole load if tightly wedged, and does not compress, if not a certain proportion of the load will be transmitted to the diagonal.

Fig. 16 is a plan of a circular dam, in this case only inside walings in short lengths, curved on the face next the piling are required, as the whole dam will be in uniform compression. When cut-waters are added to the ends of a dam as in Fig. 12, shown also in detail by Fig. 12 A, specially cut piles will be required at the corners of the sides. Figs. 12 B and 12 C are details of the methods generally used for walings and struts against sheet piles; and Figs. 17 to 20 are typical cross sections of cofferdams, illustrating the systems used for strutting and bracing them.

In some cofferdams in which the working area inside is required to be clear of struts, the sheet piling is supported by horizontal braced girders at suitable vertical intervals, and with bolted joints, these girders are subject to end compression in addition to the lateral water pressure.

Economy in Dredging

Although a great deal of time and thought has been devoted to the mechanical handling of goods by modern manufacturers, port authorities and other interested parties, little progress, if any, has been made with the mechanical handling and disposal of those unwanted products of which most ports seem to possess a surfeit—that is, silt and mud.

The steam driven mechanical ladder dredger, the most popular machine in England for raising river silt and detritus, as well as virgin soil, has been in use for over a century, and the design of the hopper employed using gravity discharge of the dredged material through hinged bottom doors is as old or older, but what progress has been made in the economy of actual raising and carriage of the spoil?

Doctor Herbert Chatley, in his paper on "Dredging Machinery," read before the Institution of Civil Engineers in 1945 (Paper No. 5450), pointed out that the largest dredger then in existence was built in 1909, and in a few cases only had diesel engines been substituted for steam plant. The Port of Antwerp, before the late war, had a very modern non-propelled dredger with buckets of approximately .75 cubic metre capacity, diesel electric driven and all electric winches. A non-propelled craft in England of similar capacity, steam driven, would need about 15-16 men for single shift working with 23-25 (both figures, of course, being according to winch gear layout) for two-shift working. The Belgian dredger had five men for single shift in dock, including the master, who conned the ship and ladder at the same time from a glass-sided cabin placed on top of the hoist frame. Main engine and separate winch controls of the contactor type were located in the cab and operated by the master. Obtaining a master capable of working this complicated tool was difficult, while the maintenance of the machinery was an even greater problem. It is significant that the experiment of diesel electric drive does not appear to have been repeated for bucket dredgers, although the Mersey Docks & Harbour Board Dredger, No. 26, a twin screw triple grab hopper dredger, described elsewhere in this issue, has been fitted with diesel electric drive.

In the discussion following Dr. Chatley's paper, a welded bucket (in place of the usual British practice of a riveted bucket) was suggested, and standardisation of the bucket chain parts was another idea. Generally, however, the bucket ladder dredger remains in external design much as when first introduced. Refinements, such as manganese steel for tumblers, bucket cutting lips, special steels for pins, bushes, etc., are recent innovations; whilst the relative merits of geared and belt drive are still debated.

Occasional experiments have been made in winch layouts in the larger dredgers to economise man power.

In the engine room the steam reciprocating engine working with Scottish multitubular single ended boiler still holds the field, as it did for so many years in our cargo ships.

The bucket chain is, and is likely to remain, the heaviest maintenance item on a ladder dredger, due to the abrasive nature of the materials the dredger has to work in.

Costs of raising and loading into hopper alongside are usually less than those of disposing of the material, and in this connection one of the most interesting points was made during the discussion. It was pointed out that large sums were spent by dredging contractors and others who carry too high a percentage of water in their cargoes. A hopper newly loaded to the coamings with fine silt or mud may carry away a spoil cargo consisting of anything up to 30% or more of water, unless given some hours standing by to drain. In a large hopper barge, conveyance of this water represents a great waste of money.

The secret of avoiding this is to obtain and keep a good seal to the hopper doors, when the hopper is light, so that when placed alongside the dredger preparatory to loading, the well, instead of being filled to the outside level, has only a small quantity of water in it sufficient to ensure reasonable stability. The advocate of this method stated that his hopper crews pumped out much of the water already in the well when the hopper was in light condition. A light, petrol driven, portable pump, carried on deck was found to be sufficient for the purpose, pumping being done during return trips from the spoil grounds.

To keep down expenditure of effort there is a real need for a simple type of bottom dooor seal that would be tight enough to prevent seepage of water into the hopper well from outside, so that pumping was not too protracted.

The money saved by this means, particularly when carrying fine silts and mud, would be considerable, and the hopper could put in more loading time at the dredger on account of the increase in capacity gained.

The Mechanical Handling Exhibition, recently held in London, had many ingenious time-saving cargo handling devices on view. Can our hopper designers and builders produce a vessel fitted with self-sealing doors, so that the wasted effort of carrying large quantities of water to the spoil grounds is avoided?

Port of London Authority

Excerpts from Annual Report for the Year ended 31st March, 1948

During the year under review, the tonnage of vessels entering and leaving the port was the highest since 1940 though only about two-thirds of the tonnages for the years immediately prior to the war. The tonnage of goods passing through the port reached a total figure equivalent to about seven-eighths of the tonnage dealt with in 1939.

Trade of the Port

Shipping. The total net register tonnage of vessels that arrived and departed with cargoes and in ballast, excluding naval vessels, during the twelve months ended 31st March, 1938-1948, was as follows:—

	Tons		Tons
1938 ...	62,949,744	1944 ...	18,703,909
1939 ...	62,085,840	1945 ...	33,353,992
1940 ...	46,070,103	1946 ...	31,017,310
1941 ...	20,114,208	1947 ...	34,811,540
1942 ...	17,529,591	1948 ...	41,372,625
1943 ...	14,665,170		

Imports and Exports. The tonnage of imported and exported goods, foreign and coastwise, of the Port of London for the twelve months Ended 31st March, 1948 and 1939, respectively, was as follows:—

	1948 Tons	1939 Tons
Imports	30,649,820	34,098,315
Exports	5,728,431	7,563,748
Total ...	36,378,251	41,662,063

Finance

The balance of Borrowing Powers unexercised at 31st March, 1948, amounted to £1,206,691.

The amount standing to the credit of the Stock (Redemption) Funds at 31st March, 1948, was £381,872. Investments held on account of these Funds stand in the books at a value of £176,734, leaving a balance of £205,138 for investment, or to be used in exercise of Borrowing Powers.

Supplementary to the statutory requirements in regard to Port Stock, provisional Redemption Funds amounted at 31st March, 1948, to £31,912.

The Capital Redemption Account now stands at £7,666,779.

Capital Expenditure. The Capital Expenditure for the year ended 31st March, 1948, amounted to £128,796.

In addition, an amount of £357,121 was expended on the repair and reinstatement of war damage.

Working Results. The following is a summary of the year's working:—

	£
Total Revenue	10,290,902
Total Expenditure	6,862,947
Balance of Revenue	3,427,955
Deduct—Interest on Port Stock; Stock (Redemption) Fund charges, Income Tax, etc.	2,884,015
Balance (Surplus) for the year	543,940
Balance (Deficit) brought forward from 31st March, 1947	549,130
Leaving to be carried forward a deficit of	£5,190

The expenditure during the year on account of the General Fund for the Maintenance and Renewal of Premises and Plant and for Dredging was £323,813, and, after transferring £350,000 from Net Revenue Account and crediting the sum of £203,000 received under Section 2 (1) (b) of the Compensation (Defence) Act, 1939, the balance standing to the credit of the Fund at 31st March, 1948, was £1,550,792. Investments held on account of this Fund stand in the books at a value of £1,320,839.

The General Reserve Fund amounts to £894,757 and is fully invested in Trustee Securities.

The amount standing to the credit of the Insurance Fund at 31st March, 1948, was £559,088. Investments held on account of this Fund stand in the books at a value of £544,149.

A further sum of £400,000 has been reserved towards meeting the deficiency which it is anticipated will arise on the next quinquennial valuation as at 31st March, 1949, of the Port of London Authority Pension Fund, making a total reserve of £900,000.

Works

The Authority was unable owing to supply and other difficulties to put in hand the entire programme of works of high priority which had been drawn up for the year ended March, 1947, as a first step in reducing the arrears of maintenance resulting from the war years. Nevertheless, it was considered desirable to prepare a further programme of such works for the year under review involving an expenditure in the neighbourhood of £475,000, but progress in its execution has been delayed for similar causes as in the previous year.

The Authority have continued as rapidly as conditions permit to proceed with the reinstatement of war-damaged transit sheds and storage accommodation.

Dredging. During the year 1,417,915 cub. yds. of material were dredged from the river and 1,188,556 cub. yds. from the docks.

General

No major variation in rates and charges has been made in the course of the year under review.

War Damage. The "Working Party" of representatives of the Dock and Harbour Authorities' Association and of the Government under the Chairmanship of Mr. Thomas Haworth, the Authority's Chief Accountant, have reached agreement on the major issues involved in connection with the war damage claims of port authorities throughout the country and the matter now awaits the promotion of legislation on the basis of their recommendations.

The Authority's provisional claim amounts to some £13,500,000 and it is anticipated that the Authority's payment in respect of war damage contribution will amount to about £780,000.

Handling of Explosives. The Authority have expressed their serious concern at the risks involved in the handling of explosives within the dock areas and have represented in appropriate quarters that all possible steps should be taken by the responsible Government Departments to bring about the discontinuance as far as possible of this practice which was introduced only as an emergency measure at the outbreak of the war in 1939.

Pilferage of Goods. The Authority's Police Force have continued to take all practicable measures to protect property and goods in the docks and a Mobile Branch which completed its first year of operation, has proved an effective adjunct to the Force in this respect.

Canteens. Canteen facilities, both static and mobile, in the docks and warehouses have been improved as far as circumstances permitted to meet the obligations placed on the Authority by the Docks (Provision of Canteens) Order, 1941. The provision of additional canteen accommodation is receiving consideration. Up to the 31st March, 1948, an expenditure of £92,354 had been incurred on buildings, alterations to existing premises, etc., for this purpose, in addition to which the loss on working for the years 1942 to 1947 amounted to £40,711, exclusive of general administration costs. The working account for the year under review shows a loss of £11,854.

Siltation in the River. In pursuance of the Authority's decision to carry out a comprehensive investigation into the problem of silting in the River and Docks, arrangements were made during the year with the Department of Scientific and Industrial Research for the Hydraulic and Water Pollution Research sections of the Department to undertake certain investigations in their respective spheres.

Notes of the Month

Increased Traffic at Rotterdam.

Towards the end of September last the 6,000th ship this year arrived at the Port of Rotterdam, a total which has been reached for the first time since Holland's liberation. In the past year the 5,900th ship was registered on December 31st. This figure compares with 11,732 ships which arrived in Rotterdam during 1939.

Landing Craft for Belgian Congo.

Ten tank-landing craft, purchased by the Belgian Authorities from the British Ministry of Supply, are shortly to start work hauling barges along the inland waterways in the Belgian Congo. The craft were dismantled at Antwerp and reconstructed in a shipyard at Leopoldville, and the first vessel has just been relaunched.

Improvements at the Port of Ankara.

It is reported from Ankara that the State Shipping Administration plans to spend between five and six million Turkish lire on improvements and extensions at the Port of Istanbul during 1949. This expenditure will be part of a comprehensive port modernisation and re-equipment scheme estimated to cost 150,000,000 Turkish lire during the next few years.

Ice Breaker for Finland.

The Finnish Government has made arrangements for the construction of an ice breaker about the same size as the "Jääkarhu," which, with the "Voima," had to be surrendered to the U.S.S.R. under the terms of the peace treaty. The new vessel is to be built at Helsinki and the Diesel propelling machinery is to be obtained from Sweden at a cost of 1,750,000 kr. The total cost of the vessel is expected to reach 750,000,000 Finnish marks and it is hoped she will be ready for service in 1952. Her propelling machinery will probably develop 10,500 h.p.

Record Ore Shipments on Great Lakes.

A new peace time ore-shipping record on the Great Lakes is expected to be established this season. At the same time there are doubts as to whether the 1948 target of 86,000,000 tons, set at the opening of the navigation season will be reached. The largest peace-time ore movement was recorded last year, when freighters moved 77,898,087 tons down the Lakes. This year's shipments are now expected to aggregate between 81,000,000 and 83,000,000 tons. Shipments up to the beginning of September totalled 50,722,890 tons compared with 46,754,812 tons a year ago.

Grain Shipments from Montreal.

Before the Port of Montreal closes for navigation early next month, an exceptionally large movement of grain is expected. As a result of a very heavy crop in Ontario, 7,000,000 bushels of Ontario wheat are being shipped through Montreal to points in Northern Europe, and another 500,000 bushels of Manitoba wheat has already been shipped. The first railway train loads of the autumn grain crop from Western Canada have also reached Montreal, but the full movement of harvest grain is not expected until more ships on the Great Lakes are available.

Facilities at the Port of Dar-es-Salaam.

The general manager of the Tanganyika Railways and Harbours recently announced that covered storage space at the Port of Dar-es-Salaam has been increased from 110,000 sq. ft. in 1939 to 190,000 sq. ft. He also stated that, at present, rail capacity is keeping pace with demands, running at about 2,400 tons a week. By the middle of next year this will be raised to at least 3,000 tons a week. The difficulties caused by the lack of railway capacity had been partially relieved by the arrival of British stock towards the end of last year, but substantial relief was not obtained until May this year, when second-hand stock from the Middle East had been received and repaired. A total of 125 33-ton bogie-wagons were now due to start arriving from the United Kingdom at the rate of about 12 a month.

Trinity House Deputy Master.

Captain Sir Arthur Morrell, the deputy master of the Corporation of Trinity House, retired from active service on October 1st; and Captain Gerald Curteis, R.N. (retd.), has been appointed deputy master in his stead.

Jarrow Oil Storage Depôt.

The Shell-Mex & B.P., Ltd., oil storage installation at Jarrow has recently been considerably enlarged. The additions have increased the storage capacity by 36,000 tons and the total capacity of the tanks is now nearly 103,000 tons. New railway sidings are also being constructed.

Belfast Harbour Commissioners' New Chairman.

At a recent meeting of the Belfast Harbour Commissioners, Mr. Kenneth D. L. Sinclair, was selected as their chairman in succession to Mr. John McCaughey who has retired. Mr. Sinclair has been a member of the Board for 14 years, and announcing the selection Mr. McCaughey said Mr. Sinclair's grandfather was also at one time a chairman of the Board.

New Lifeboat Station for Solway Firth.

In order to increase the protection of shipping in the Solway Firth the Royal National Life-Boat Institution has decided to establish another station on the Cumberland coast. It will be at Workington, and Mr. J. Z. Bridgewater, of the Workington Harbour and Docks Board, has undertaken to act as the honorary secretary. The station will be opened for a year as an experiment, and the Institution hopes to send a powerful motor lifeboat from its reserve fleet there early this month.

Extensions at the Port of Bilbao.

Among the many improvements planned for the Port of Bilbao, Spain, is the construction of the Deusto Canal, which it is expected will take some ten years to complete and will cost approximately Ptas. 100,000,000. The canal, which will be from 100 to 130 metres wide and 1,440 metres long, is designed to enable ships of up to 10,000 tons to reach the centre of the city, and the section of the River Nervion thus by-passed will serve as an inner harbour for barges and fishing vessels. Quays, 60 metres wide and having a total length of 1½ km. will also be constructed and will be equipped with an extensive railway system to link the warehouses with the main railway. Other improvement projects include the construction of a port railway station at Vega de San Mames, new workshops and warehouses, a new dry dock for larger ships at Sestao near Benedicta, a two-storey swing bridge to carry a railway and a road, quays to accommodate ocean-going liners at Portugalete and the electrification of the Bilbao-Miranda line, which is the port's only rail communication with the interior. Work on the fishing harbour at Santurce and on certain of the installations for the outer port is already in hand and it is hoped the provision of these greatly improved port facilities will accelerate the industrial development of the Bilbao region.

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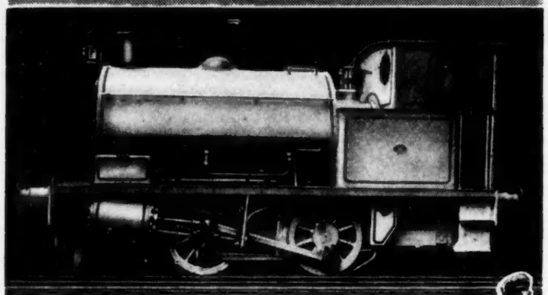

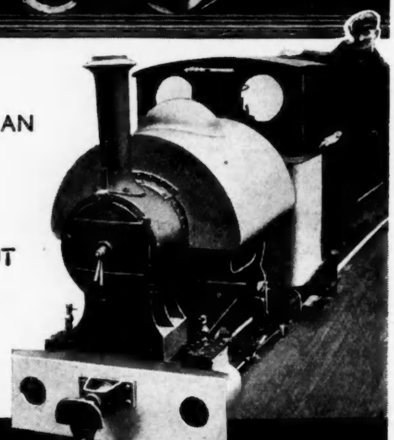

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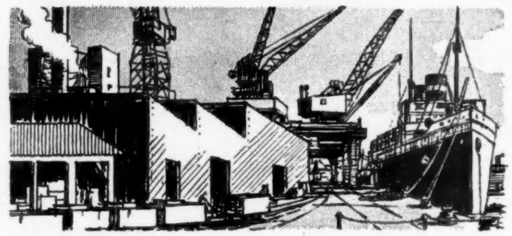

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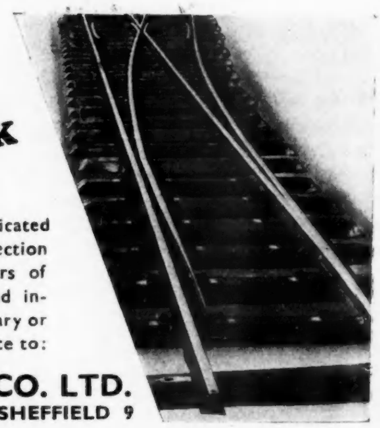

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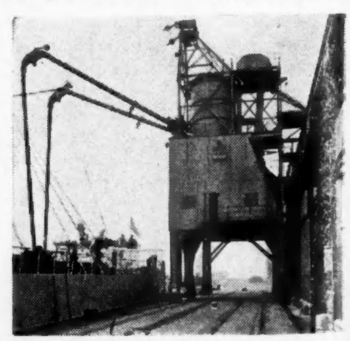

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